İSTANBUL TECHNICAL UNIVERSITY ★ INSTITUTE OF SCIENCE AND TECHNOLOGY

SOIL NAILING AND STABILITY OF SOIL NAILED SLOPES

M.Sc. Thesis by
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Department : **Civil Engineering**

Programme: **Geotechnical Engineering**

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LIST OF SYMBOLS

$\mathbf{A_s}$	: Cross-sectional area of the nail
A	: Design seismic coefficient
$\mathbf{A_{pk}}$	: Peak ground acceleration
$\mathbf{k_h}$	: Coefficient of horizontal ground acceleration
$\mathbf{k_v}$	: Coefficient of vertical ground acceleration
c	: Cohesion of the soil
$\mathbf{c_m}$	: Mobilized cohesion
$\mathbf{c_u}$	: Undrained cohesion of the soil
$\mathbf{D_a}$	: Equivalent diameter for driven nails
$\mathbf{D_c}$	: Borehole diameter for grouted nails
E	: Youngs' modulus of the nail
FS	: Global Safety Factor
$\mathbf{FS_p}$	: Safety factor with respect to pull-out
$\mathbf{FS_m}$	: Factor of safety with respect to plastic bending moment
$\mathbf{F_1}$	: Unit skin friction
$\mathbf{f_y}$	: Yield stress of the reinforcement
H	: Height of nailed wall
I	: Moment of inertia of the nail
$\mathbf{K_a}$	: Active earth pressure coefficient
$\mathbf{K_h}$	: Modulus of lateral soil reaction
L	: Length of nails
$\mathbf{L_a}$	: Adherence length of reinforcement in the passive zone
$\mathbf{L_0}$	: Transfer length of the nail
M	: Bending moment in the nail
$\mathbf{M_p}$	: Plastic bending moment of the nail
N	: Bending stiffness parameter
p	: Pressure on the nail
$\mathbf{p_1}$	: Limit pressuremeter pressure
$\mathbf{R_c}$	: Shear resistance of the nail
$\mathbf{R_n}$	: Tension resistance of the nail
S	: Nail length in the active zone
$\mathbf{S_h}$	: Horizontal spacing of nails
$\mathbf{S_v}$	: Vertical spacing of nails
$\mathbf{T_b}$	: Elastic limit of the reinforcement
$\mathbf{T_c}$	: Shear force in the nail
$\mathbf{T_m}$	: Limit shear force per unit length of nail
$\mathbf{T_{max}}$	: Maximum tension force in the nail
$\mathbf{T_n}$	: Tensile force (or axial force)in the nail
$\mathbf{T_p}$	: Ultimate skin friction force
ϕ	: Internal friction angle
$\boldsymbol{\sigma_n}$	: Average normal stress on L_a
$\boldsymbol{\phi_m}$	: Mobilized internal friction angle

τ : Lateral shear stress
τ_{mob} : Mobilized lateral shear stress
κ : Ratio of the horizontal and vertical stresses
γ : Unit weight of the soil
γ_n : Natural unit weight of the soil
δ_h : Horizontal displacement at top of wall facing
δ_0 : Horizontal surface displacement behind the wall
δ_v : Vertical displacement at top of wall facing
Γ_m : Partial safety factor
$\Gamma_{m\phi}$: Partial safety factor on friction angle
Γ_{mc} : Partial safety factor on cohesion
Γ_{mF1} : Partial safety factor on frictional resistance
Γ_{mRn} : Partial safety factor on tensile strength
Γ_{ms3} : Method coefficient
Γ_G, Γ_Q : Load factors

ZEMİN ÇİVİLEMESİ VE ZEMİN ÇİVİLİ ŞEVLERİN STABİLİTESİ

ÖZET

Son otuz yıldır, zemin çivilemesi yöntemi, özellikle Avrupa'da, kazı yüzeylerinin desteklenmesi ve şev stabilitesinde kullanılmaktadır. Bugüne kadar metal donatıların ve yüzey kaplaması teknolojisinin eksikliğinden dolayı zemin çivileri daha çok geçici dayanma yapılarında kullanılmaktaydı. Teknolojik gelişmelerle, son yıllarda bu noksanlıkların üstesinden gelinmiştir.

Çelik çubuk veya diğer metalik elemanlardan oluşan çiviler pasif donatı olarak adlandırılmaktadır. Zemin çivilemesinde kullanılan çiviler çakma çiviler, enjeksiyonlu çiviler, jet enjeksiyonlu çiviler, ve korozyon tehlikesine karşı kapsüllü çiviler olarak sınıflandırılabilirler.

Tamamlanmış bir zemin çivili duvarda, tek gözüken kısım yüzey kaplamasıdır. Kaplamanın fonksiyonları sırası ile takviyeler arasındaki lokal zeminin stabilitesini sağlamak, kazı sonrası ani gerilme boşalımını dolayısıyla ayrışmayı önlemek ve mevcut zemini erozyon ve aşınma etkilerine karşı korumaktır. Uygulamaya bağlı olarak kaynaklı çelik ağ, şotkrit, prefabrike beton, ve yerinde kalıba döküm betonarme kaplamalar kullanılmaktadır.

Zemin çivilemesi metodu granüler ve kohezyonlu zeminlerde ve heterojen birikintilerde uygulanmaktadır.

Zemine çivilenmiş yapıların tasarımına yönelik birçok güncel metot mevcuttur. Bunlar, Fransız, Alman, Davis ve Kinematik Metotlardır. Bu metotlardan ilk üçü limit denge analizine dayanırken, sonuncusu çalışan kuvvet analiz yaklaşımını içerir. Metotlarda kayma yüzeyi bi-lineer, parabolik, dairesel ya da log-spiral olarak kabul edilir. Stabilite analizleri ile kayma yüzeyini kesen takviyelerin, limit kesme, çekme ve sıyrılma kapasiteleri araştırılır.

Bu tez çalışmasında, zemin çivileme tekniği ve zemin çivili dayanma yapılarının tasarımı incelenmiştir. TALREN 97 bilgisayar programı ile zemin çivili şevlerin stabilitesi bulunmuştur. TALREN 97, TERRASOL tarafından geliştirilmiş,

geoteknik yapılarında potansiyel kayma yüzeyi boyunca stabilite analizi yapan bilgisayar programıdır.

SUMMARY

Soil nailing has been used in a variety of civil engineering projects in the last three decades, mainly in Europe, to retain excavations and stabilize slopes. To date, soil nailing has been primarily used for temporary retaining structures. This is mainly due to the engineering concerns with regard to durability of metallic inclusions in the ground and shortcomings of facing technology. In recent years, technological developments overcome these limitations.

Nails which are steel bars or other metallic elements are commonly referred to as “passive” inclusions. Steel reinforcement inclusions currently used in soil nailing process can be classified as driven nails, grouted nails, corrosion protected nails and jet-grouted nails.

The only visible part of the completed work is wall facing. The facing functions to ensure local ground stability between reinforcements, limit decompression immediately after excavation and protect the retained soil from surface erosion and weathering effects. Depending on the application welded wire mesh, shotcrete, precast concrete or cast in place concrete facings has been used.

Soil nailing method has been used both granular and cohesive soils and relatively heterogeneous deposits.

There are several methods currently available for the design of nailed soil structures. These are French Method, German Method, Davis Method and Kinematical Method. The first three methods are based on limit equilibrium analysis, where the last is based on working stress analysis. These methods assume the failure surface to be bi-linear, parabolic, circular or log-spiral. The variable limit shearing, tensile, and pull-out resistances of the reinforcements crossing the failure surface are considered in the stability analysis.

In this study, the soil nailing technique and design of soil nailed retaining structures are examined. A computer program, TALREN 97, has been applied to evaluate the stability of reinforced slopes. TALREN 97, developed by TERRASOL, is a stability analysis program for geotechnical structures along potential failure surfaces.

1.INTRODUCTION

The development of retaining structures received a new impetus in 1958 through the introduction of ground anchors. High and relative slender walls as pile, diaphragm and sheetpile walls, could now be constructed prior to excavation and tied back with ground anchors during excavation. A new type of retaining structure – the element wall – was developed about 10 years later. Pre-cast or in-situ-cast concrete elements were placed checkerboard-like onto the excavated soil surface and tied back with anchors [8].

A new idea was born at the end of the sixties. Gravity walls constructed with artificially placed soils and strengthened with steel reinforcement could replace anchored structures. This method, known as reinforced earth, became very economical, since soil is used for the main part of the structure. The disadvantage of this method is that the retaining wall has to be built from bottom to top, which means that the full excavation has to be completed in advance of the construction of the wall [8].

The consequent criticism of this idea led to the method of soil nailing in the beginning of the seventies. Instead of constructing the wall from bottom to top the opposite way was taken. The natural in-situ soil was used for the gravity wall. Together with the proceeding excavation, which was carried out in steps of 1m to 1,5m, the soil was reinforced with steel bars, called nails [3].

Today the technique of soil nailing is far spread and advanced in Germany, France, Great Britain, Japan and the United States [8]. The fundamental concept of soil nailing consists of reinforcing the ground by passive inclusions, closely spaced, to create in-situ a coherent gravity structure and thereby to increase the overall shear strength of the in-situ soil and restrain its displacements. Reinforcing elements are installed by placing them into the existing soil slope or new excavation. The basic design consists of transferring the resisting tensile forces generated in the inclusions into the ground through the friction mobilized at the interfaces. It should be noted that these systems allow the engineer to efficiently use the in-situ ground to provide vertical or lateral structural support. They present significant technical advantages over conventional rigid gravity retaining walls or external bracing system that result in substantial cost savings and reduced construction periods. Therefore, they are increasingly used in civil engineering projects [5].

To date, soil nailing has been primarily used for temporary retaining structures. This is mainly due to the engineering concerns with regard to durability of metallic inclusions in the ground and shortcomings of facing technology. In recent years, technological developments have included low cost corrosion protected nails, innovative installation techniques such as jet nailing and nail launching as well as prefabricated concrete or steel panels to overcome these limitations. Soil nailing has now become a common construction technique for a wide variety of engineering applications including: stabilization of railroad and highway cut slopes, excavation retaining structures in urban areas for high-rise building and underground facilities, tunnel portals in steep and unstable stratified slopes, construction and retrofitting of bridge abutments, and other civil and industrial projects [5].

In this study, the soil nailing technique and design of soil nailed retaining structures are examined. A computer program, TALREN 97, has been applied to evaluate the stability of reinforced slopes. TALREN 97, developed by TERRASOL, is a stability analysis program for geotechnical structures along potential failure surfaces. The program considers hydraulic and seismic data, in addition to various types of soil inclusions (nail, anchor, brace, reinforcing strip, geotextile, pile, micropile, sheetpile, etc.) [18].

2.SOIL NAILING TECHNIQUE

2.1 DESCRIPTION OF SOIL NAILING

A soil nail is a structural element which provides load transfer to the ground. The basic concept of soil nailing is to reinforce and strengthen the existing ground by installing closely spaced steel bars, called “nails” (Figure 2.1), into a slope or excavation as construction proceeds from the “top down” [3]. Nails work in tension but are considered by some to work also in bending/shear. Nails are commonly referred to as “passive” inclusions. The term “passive” means that the nails are not as tiebacks when they are installed. The effect of the nail reinforcement is to improve stability by [2],

a. increasing the normal force and for this reason increasing the soil shear resistance along potential slip surfaces in frictional soils.
b. reducing the driving force along potential slip surfaces in both frictional and cohesive soils.

2.2 CONSTRUCTION SEQUENCE OF A SOIL NAILED WALL

The following is the typical sequence to construct a soil nail wall (Figure 2.2) [3];

1. Excavate a Small Height Cut
2. Drill Hole for Nails
3. Install and Grout Soil Nail Tendon
4. Place Geocomposite Drain Strips
5. Place Initial Shotcrete Layer
6. Install Bearing Plates and Nuts
7. Repeat Process to Final Grades
8. Place Final Facing

Note : Order of nail and shotcrete installation may be reversed.

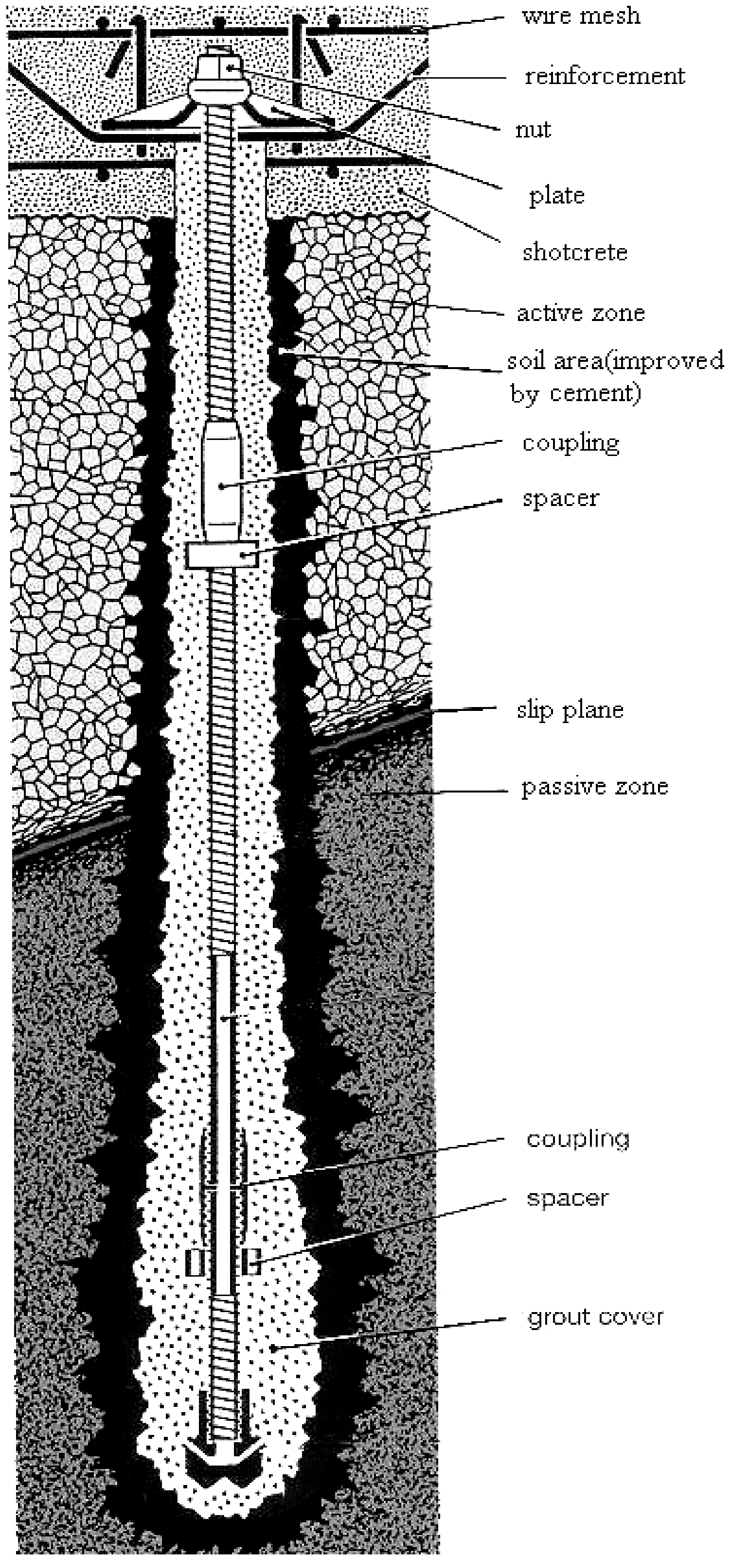


Figure 2.1 Typical soil nail [19]

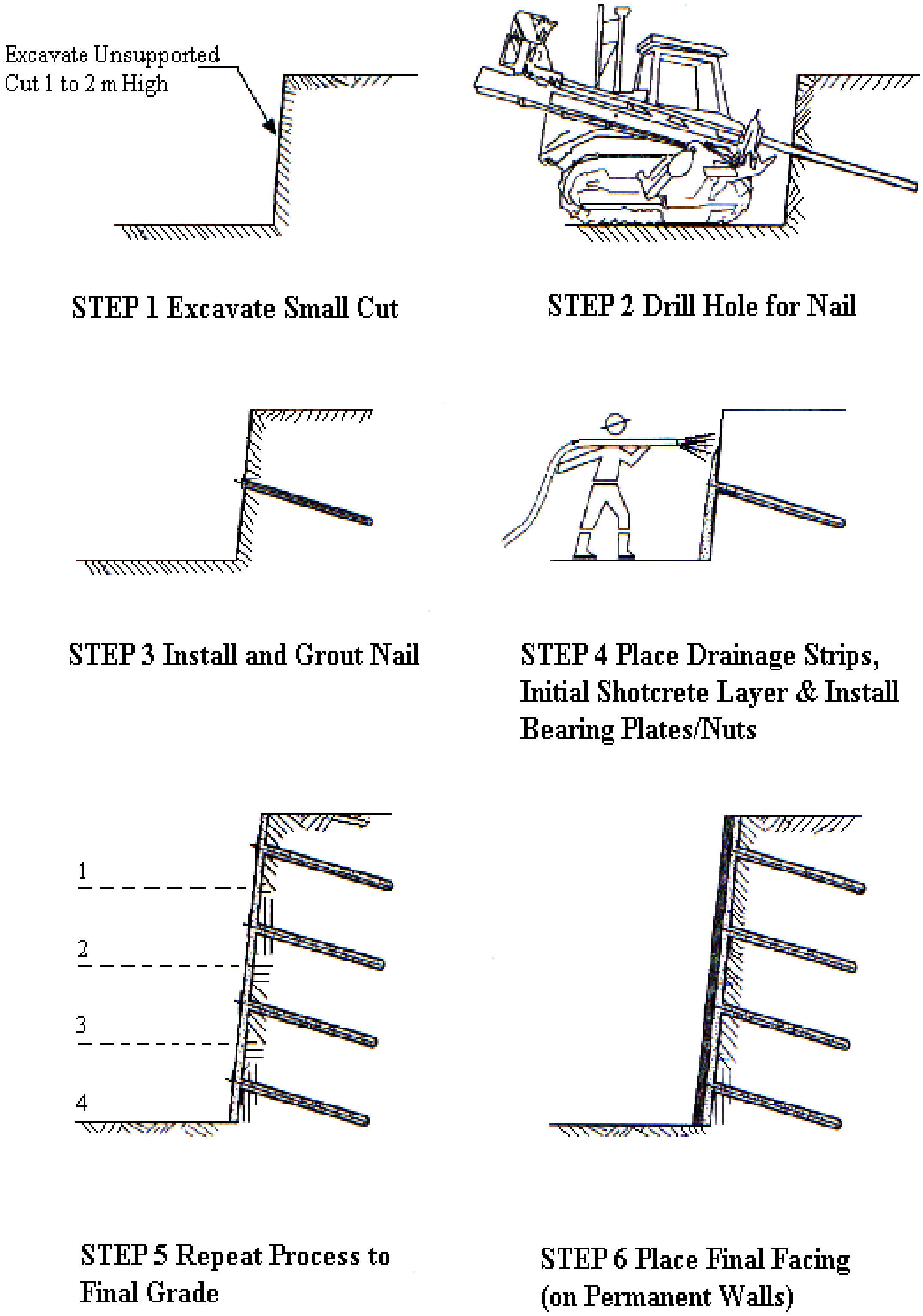


Figure 2.2 Typical nail wall construction sequence [3]

2.3 ADVANTAGES OF SOIL NAILING

Soil nailing cannot replace all other methods of retaining structures, neither technically nor economically, but it has several advantages [1,2,8]

1. Only light construction equipment is required to install nails as well as simple grouting equipment. Grouting of the boreholes is generally accomplished by gravity.
2. The method is very economical, if
   a. It is not possible to use large machines
   b. The geometry of the wall is complex
   c. There is little space for the construction.
3. The nails consist of low-strength steel. Thus the problem of corrosion protection is extremely reduced compared to the use of permanent anchors.
4. The bottom of the wall is equal to the depth of the excavation. This saves a lot of material.
5. The failure mode is good-natured, i.e. the retaining structure does not collapse suddenly and without large deformation.
6. The construction may be carried out with little environmental disturbance, which means little noise and hardly any vibration.
7. Since there are large number of nails, failure of any one may not detrimentally affect the stability of the system, as would be the case for a conventional tieback system.
8. Surface deflections can be controlled by the installation of additional nails or stressing in the upper level of nails to a small percentage of their working loads.
9. In heterogeneous soils with cobbles, boulders and weathered zones or hard rock zones, it offers the advantage of small diameter shorter drill holes for nail installation and eliminates the need for soldier pile installation.

2.4 LIMITATIONS OF SOIL NAILING

Soil nailing also has disadvantages [1,2,8]:

1. The horizontal deformations of the wall may reach the order of 0,2 to 0,4% of the wall height and are usually larger than those of anchored structures.

2. Without additional measures soil nailing can not be used for underpinning of large buildings.
3. The aesthetic form of the wall face with plain shotcrete is not satisfying. Additional measures have to be taken, e.g. covering with pre-cast elements or greening with plants.
4. The long term performance of shotcrete facings has not been fully demonstrated particularly in areas subject to freeze-thaw cycles.
5. Groundwater drainage systems may be difficult to construct and their long-term effectiveness is difficult to ensure.
6. Permanent underground easements may be required.

2.5 COMPARISON WITH PRESTRESSED GROUND ANCHORAGES

There would appear to be a number of similarities between nails and prestressed ground anchorages when used for slope or excavation stability. Indeed it is tempting to regard nails merely as “passive” small scale anchorages. There are major functional distinctions to be made [14];

a. Ground anchorages are stressed after installation so that in service they ideally prevent any structural movement accurring. In contrast, soil nails are not prestressed and require very small soil deformation to cause them to work.

b. Nails are in contact with the ground over most of their length (typically 3 to 10m), whereas ground anchorages transfer load only along the distal, fixed anchorage length. A direct consequence of this is that the distribution of stressed in the retained mass is different for each type.

c. Since nails are installed at a far higher density (typically 1 per 0,5 to 5 m^2) the consequences of a one unit failure are not necessarily so severe.

d. As high loads have to be applied to anchorages, appropriate bearing facilities must be provided at the head to eliminate the possibility of “punching” through the facing of the retained structure. Substantial bearing arrangements are not necessary with nails whose low individual head loadings are easily accommodated on small steel bearing plates place on the shotcreted surface.

e. Individual anchorages tend to be longer (15 - 45m) and so many necessitate larger scale installation equipment.

2.6 COMPARISON WITH REINFORCED EARTH WALLS

Although soil nailing shares certain features with the older and more widely known technique of reinforced earth for retaining wall construction, there are also some fundamental differences which are important to note.

The main similarities are [14]:

a. The reinforcement is placed in the soil unstressed; the reinforcement forces are mobilized by subsequent deformation of the soil.

b. The reinforcement forces are sustained by frictional bond between the soil and the reinforcing element. The reinforced zone is stable are resist the thrust from the unreinforced soil it supports; like a gravity retaining structure.

c. The facing of the retained structure is thin and does not play a major role in the overall structural stability.

The main dissimilarities are [14]:

a. Although at the end of construction the two structures may look similar, the construction sequence is radically different. Soil nailing is constructed by staged excavations from “top-down” while reinforced earth is constructed “bottom-up”, (Figure 2.3). This has an important influence on the distribution of the forces which develop in the reinforcement, particularly during the construction period.

b. Soil nailing is an in-situ reinforcement technique exploiting natural ground, the properties of which can not be preselected and controlled as they are for reinforced earth fills.

c. Grouting techniques are usually employed to bond the reinforcement to the surrounding ground: load is transferred along the grout to soil interface. In reinforced earth, friction is generated directly along the strip to soil interface.

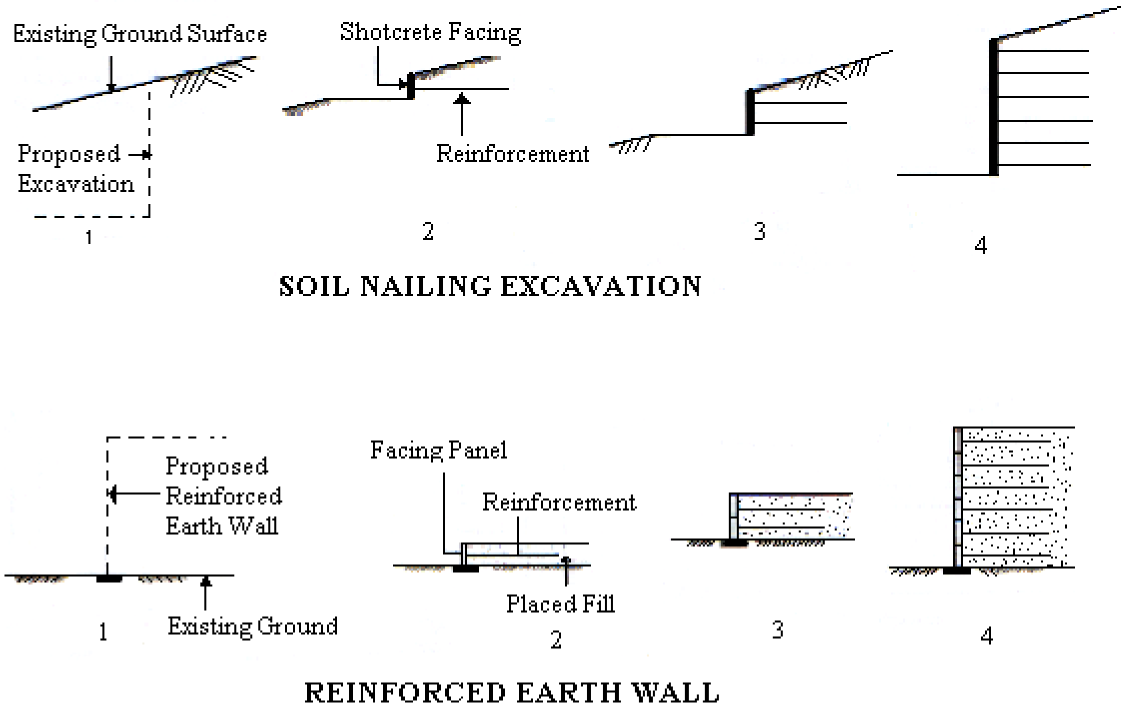


Figure 2.3 Contrast of the construction sequence (a) "top down" in soil nailing and (b) "bottom up" for reinforced soil [14]

2.7 CONSTRUCTION MATERIALS

The materials required include the nails themselves and the associated corrosion protection systems, the nail grout, the drainage materials, the shotcrete/concrete facing materials, and the system for providing a construction between the nail head and the facing.

2.7.1 Nails

The nails used in soil nailing resisting structures are generally steel bars or other metallic elements that can resist tensile stresses, shear stresses, and bending moments. Conventionally, the steel reinforcing elements used for soil nailing can be classified as (a) driven nails and (b) grouted nails. However, specially designed corrosion-protected nails have also been used in permanent structures, specifically in aggressive environments. During the past decade the most significant technological innovations have been the development and use of the jet-grouted nails (Louis, 1986) and the launched soil nails. A brief description of the available nailing systems is outline below [1-5,20]:

a. Driven Nails :

Driven nails are suitable for temporary construction. They are small-diameter (15 to 46 mm) rods or bars, or metallic sections, made of mild steel with a yield strength of 350MPa. They are closely spaced (2 to 4 bars per square meter) and create a rather homogeneous composite reinforced soil mass. The nails are driven into the ground at the designed inclination using a vibropercussion pneumatic or hydraulic hammer (Figure 2.4) with no preliminary drilling. Special nails with an axial channel can be used to allow for grout sealing of the nail to the surrounding soil after its complete penetration. This installation technique is rapid and economical (4 to 5 per hour). However, it is limited by the length of the bars (maximum length about 20m) and by the heterogeneity of the ground.

Figure 2.4 Nails are driven into the ground at the designed inclination using a vibropercussion pneumatic or hydraulic hammer with no preliminary drilling [24]

b. Grouted Nails :

Grouted nails are suitable for temporary construction and, where soils are not highly corrosive (Figure 2.5). They are generally steel bars (15 to 46 mm in diameter) with a yield strength of 420 MPa. They are placed in boreholes (10 to 15 cm in diameter) with a vertical and horizontal spacing varying typically from 1 to 2 m depending on the type of the in-situ soil. The nails are usually cement-grouted by gravity or under low pressure. The nail grout consists of a neat cement grout with a water-cement ratio of about 0,4 to 0,5. Sand-cement grout may also be used in conjunction with large nail holes for economic reasons. Ribbed bars can be used to improve the nail-grout adherence, and special perforated tubes have been developed to allow injection of the grout through the inclusion. For permanent applications, nails may be epoxy-coated or provided with a protective sheath for corrosion protection.

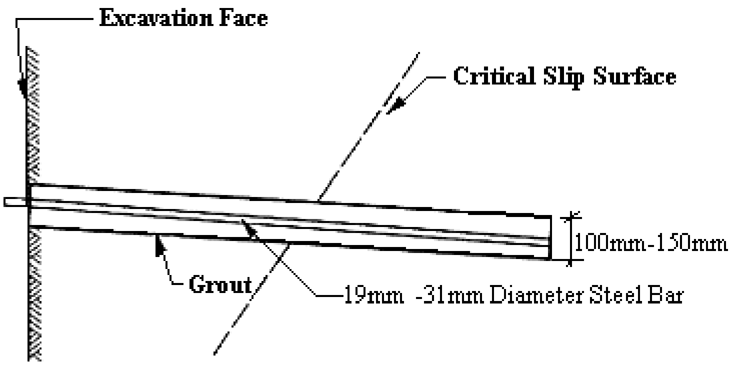


Figure 2.5 Grouted Soil Nail [20,25]

c. Corrosion Protected Nails :

The steel bar is protected against corrosion by either an epoxy or by encapsulation within a cement grout-filled plastic sheathing. Each of these measures results in isolating the tendon from the corrosive environment to varying degrees.

♦ Encapsulated Corrosion Protection :

"Encapsulated" corrosion protection must commonly consists of encasing the tendon in a grout filled corrugated PVC (poly-vinyl chloride) or HDPE (high density polyethylene) tube. The annular space between the tendon and the corrugated tube, commonly specified as a minimum of 5 mm, is filled with neat cement grout. Internal spacers are used to achieve the specified grout cover inside the encapsulation. Encapsulated corrosion protection is often referred to as “double” corrosion protection.

♦ Epoxy Corrosion Protection :

Epoxy corrosion protection consists of a fusion-bonded epoxy coating applied to the tendon. The minimum required thickness of epoxy coatings is 0,3 mm. Bearing plates and nuts that will be uncased in a structural wall facing will be protected by the concrete cover, and typically are not epoxy coated.

d. Jet-Grouted Nails

Jet-grouted nails are composite inclusions made of a grouted soil with a central steel rod, which can be as thick as 30 to 40 cm. The nails are installed (Figure 2.6) using a high frequency (up to 70 Hz) vibropercussion hammer, and cement jet grouting is performed during installation. The inner nail is protected against corrosion using a steel tube. The jet-grouting installation technique provides improvement of the surrounding ground and increases significantly the effective nail diameter and the pull-out resistance of the composite inclusion providing effective means for constructing soil nailed structures in clayey soils. Table 2.1 presents typical grouted nail diameter and ultimate pull-out capacity values for different types of soils.

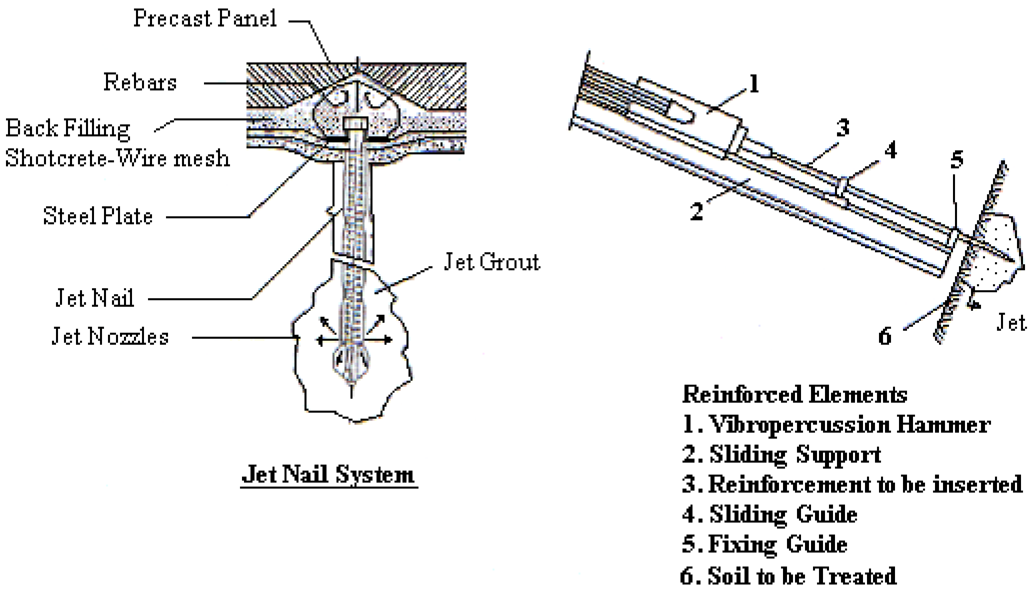


Figure 2.6 Jet Nailing [5]

Table 2.1 Typical grouted nail diameter and ultimate pull-out capacity values for different soil types [5]

Ground	Gravel	Sand	Silt	Clay
Bulp Diameter [cm]	60	40	30	20
Ultimate Pullout Resistance [kN/m]	1275	555	210	75

e. Launched Nails

The nail launching technology consists of firing directly into the ground, using a compressed air launcher, nails of 25 mm and 38 mm in diameter, made from bright bar with nail lengths of 6 meters or more. The nails are installed at speeds of 200 mph with an energy transfer of up to 100 kJ. This installation technique enables an optimization of nail installation with a minimum of site disruption (Figure 2.7). During penetration the ground around the nail is displaced and compressed. The annulus of compression developed reduces the surface friction and minimizes damage to protective coatings such as galvanized and epoxy. The technology is presently used primarily for slope stabilization although successful applications have also been recorded for retrofitting of retaining systems. However, a rigorous evaluation of the pull-out resistance of launched nails is required prior to their use in retaining structures.

Figure 2.7 Soil Nail Launcher mounted on a hydraulic excavator [26]

2.7.2 Drainage Systems

Ground water is a major concern in both the construction of soil nail retaining walls and in their long-term performance. Soil nail walls are best suited to applications above the water table. In order to protect the structures against the effects of water, some provisions for drainage must be taken. Typical soil nail wall drainage systems include; Surface Collector Ditches, Geotextile Face Drains, Shallow PVC Drain Pipes, Weep Holes and Horizontal Drains [3].

a. Surface Collector Ditch :

It is a recommended element for controlling surface flows. Where larger graded slope areas exist above the wall, installation of plastic film slope protection sheeting above the collector ditch provides another quick and inexpensive means of controlling surface water during construction (Figure 2.8) [4].

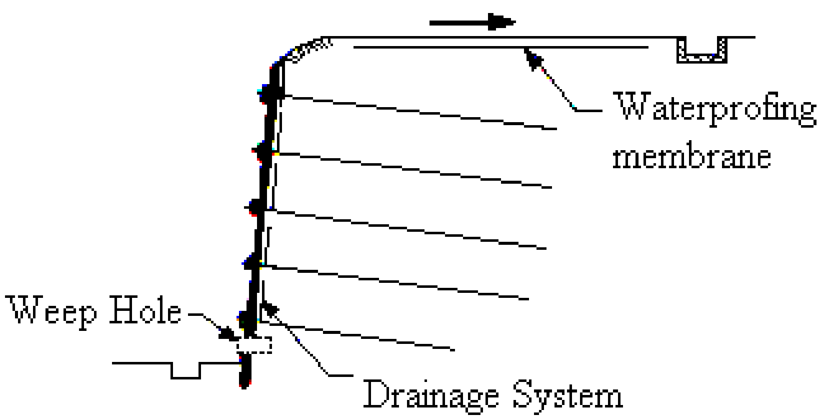


Figure 2.8 Protection Against Surface Waters [4]

b. Geotextile Face Drains :

These are 400 mm wide prefabricated geotextile drain strips, and are centered between the vertical nail columns (Figure 2.9). The strips are connected to weep hole outlet pipes and to a footing drain at the wall base. Drainage strips are used where small quantities of water are present. They may not be suitable where large quantities of groundwater are encountered.

c. Shallow PVC Drain Pipes (Weep Holes) :

These are typically 300 to 400 mm long, 50 to 100 mm diameter PVC pipes located where heavier seepage is encountered [3].

Typical permanent face drain configurations for geotextile drain strips discharging either into toe drains through weep holes in the facing are shown on figure 2.9.

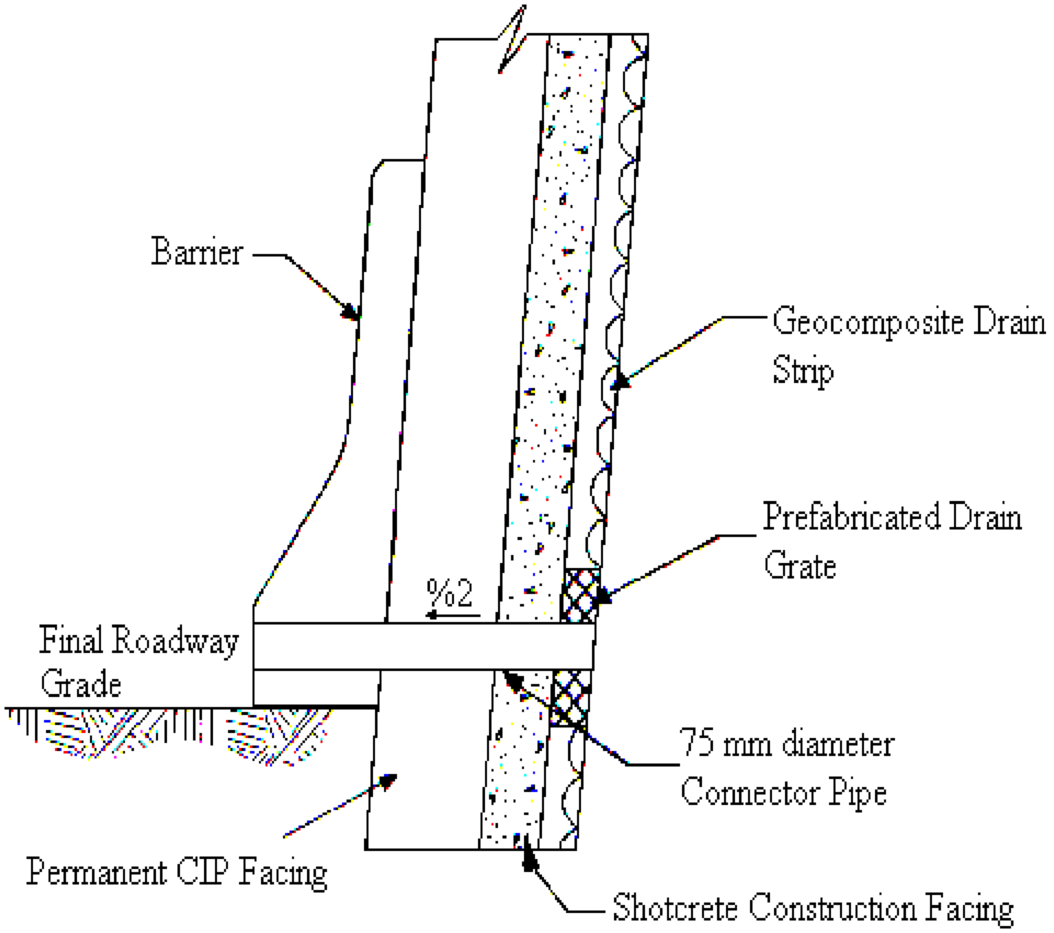


Figure 2.9 Typical Weep Hole Drain [3]

d. Horizontal Drains:

Deep horizontal drains, typically consisting of 100 mm diameter tubes (Figure 2.10) and inclined upward at 5 to 10 degrees to the horizontal (Figure 2.11). The design spacing and depth of these drains are site specific, but they will typically be longer than the length of the nails. Deep horizontal drains may also be used to control unanticipated water flow during construction.

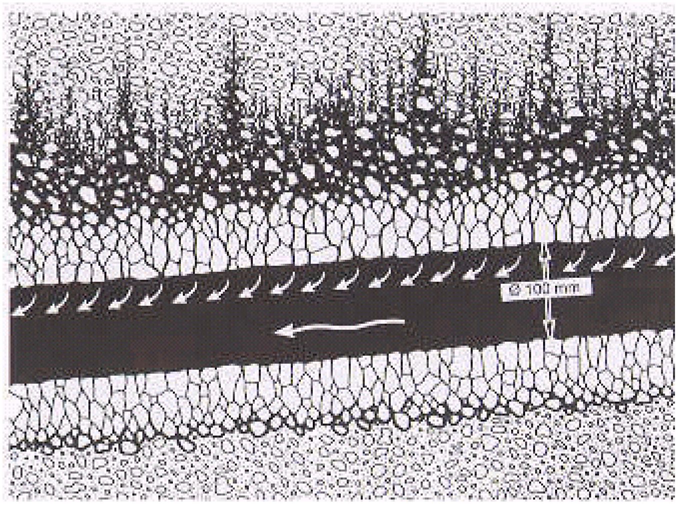


Figure 2.10 The Diameter of the Horizontal Drains [31]

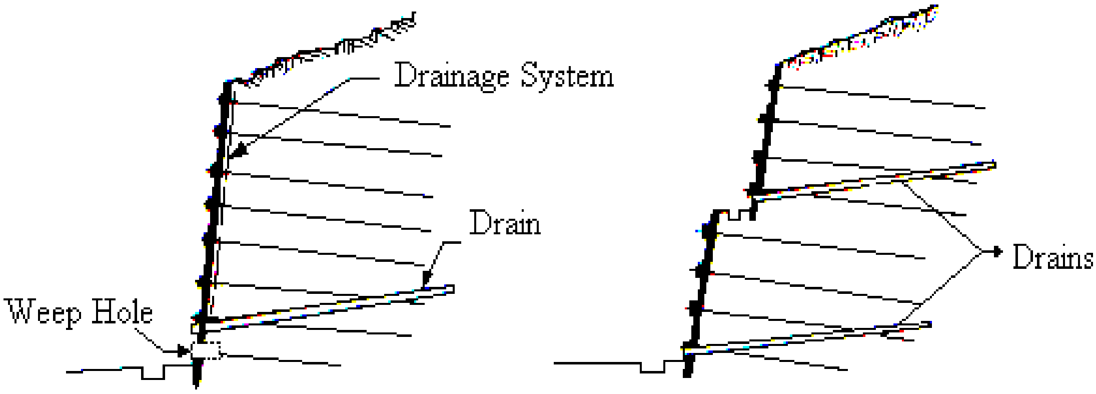


Figure 2.11 Protection Against Groundwater [4]

2.7.3 Wall Facings

The facing of the soil nailed structure is not a major structural load carrying elements. Structural wall facing protects the retained soil against weathering and erosion, and resisting lateral earth pressure. The facing consists of two component parts which are the "construction facing" and "final facing"[2,3,5]. This is defined primarily in terms of the timing of construction (Figure 2.12).

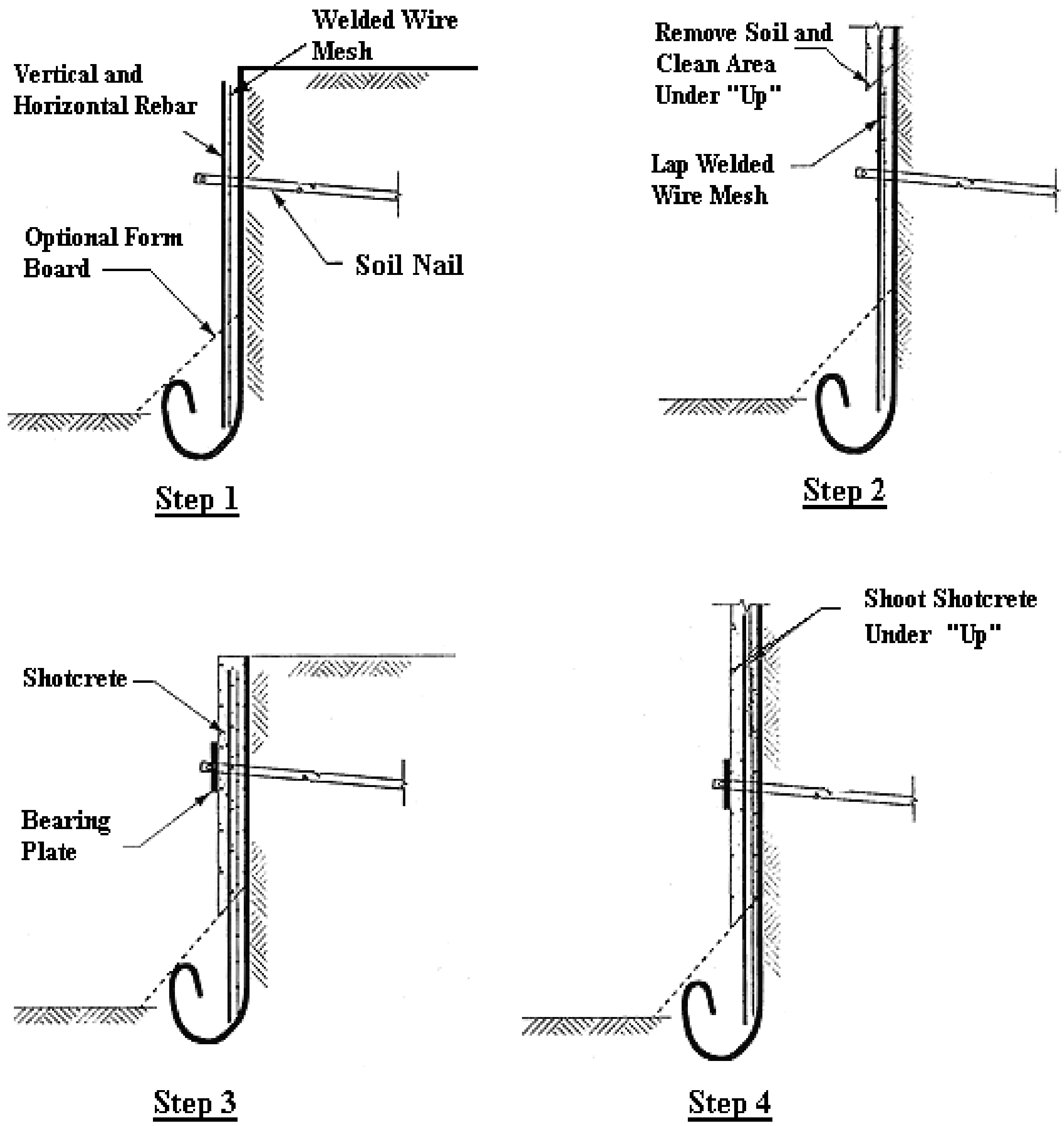


Figure 2.12 Soil Nail Wall Facing Construction Sequence [20]

2.7.3.1 Construction Facing

The “construction facing” is the facing erected during excavation. It is an initial construction of the wall and is most commonly a minimum 100 mm thick mesh-reinforced wet-mix shotcrete [3]. This system provides a continuous, flexible surface layer over the excavated soil face.

2.7.3.1.1 Shotcrete Facing

a. The Function of Shotcrete in Soil Nailing:

The function of shotcrete in soil nailing is both to transfer the earth pressure reaching the wall face from the soil to the nails and to prevent deterioration of the excavated soil face (Figure 2.13). Shotcrete is usually applied soon after excavation of a lift and placement of nails, but may also be applied before nail installation. The shotcrete must restrict the movement of the surrounding ground and be able to adapt to some ground movement .

From a quality perspective, the construction facing is less critical than the permanent facing, except from worker safety perspectives. Because it is the backing for the permanent facing, final quality of the construction facing shotcrete is important only the degree that it will not degrade excessively due to aggressive groundwater or freezing and thawing will protect embedded steel from corrosion, and will retain integrity around the nail head plates.

Figure 2.13 Shotcrete Stabilized Soil Nailed Wall [23]

b. Types of Shotcrete :

There are two methods of placing shotcrete (Figure 2.14); the wet-mix and dry-mix processes. In dry mix, aggregate and cement are blended and deposited in the gun, the mix water is added at the nozzle and is therefore instantaneously adjustable at the work face, the material is conveyed by compressed air from the gun through the nozzle (Figure 2.15). In wet-mix, a plastic mix of aggregate, cement water and admixtures are conveyed to the nozzle by hydraulic pump and nozzle velocity is achieved by compressed air (Figure 2. 16).

Figure 2.14 Placing Shotcrete [23]

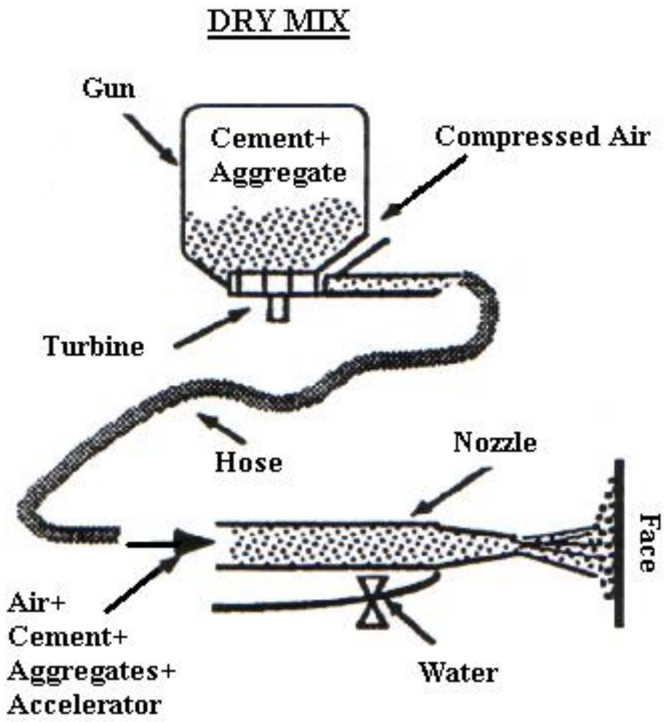


Figure 2.15 Dry-Mix Process [3]

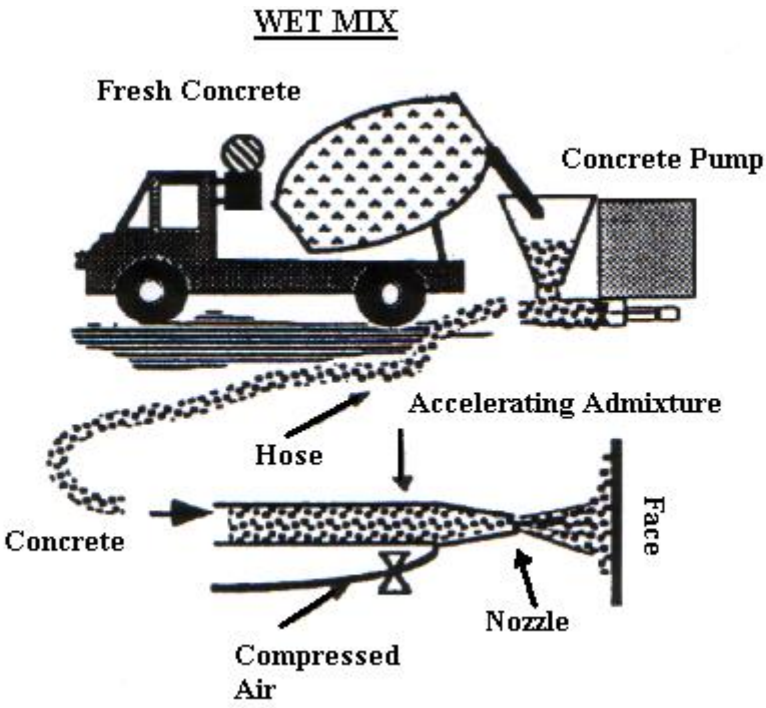


Figure 2.16 Wet-Mix Process [3]

The wet-mix shotcrete process is preferred to the dry-mix process and is used almost exclusively for soil nail wall facings. The advantages of the wet-mix process include better quality control of the water content (water-cement ratio of about 0,45 to 0,50), the ability of air-entrain for improved freeze-thaw durability, and ready availability from local ready-mix plants. Also wet-mix is generally simpler, faster and more economical. A brief comparison of the processes is given in table 2.2 [1].

Table 2.2 Comparison of operational features of dry and mix processes [1]

DRY MIX	WET MIX
Mixing water and consistency of mix are controlled at nozzle.	Mixing water controlled at delivery equipment and can be accurately measured.
Better suited for mixes containing light-weight porous aggregates.	Better assurance that the mixing water is thoroughly mixed with other ingredients. This may result in less rebound and waste.
Capable of longer hose lengths	Less dust accompanies the gunning operation.

c. Shotcrete Materials :

Shotcrete may include the following materials [3];

♦ *Cement* :

Portland cement of all types are used in shotcrete.

♦ *Aggregate :*

A commonly used gradation specification for soil nailing shotcrete is given in table 2.3.

♦ *Reinforcing Steel*

Table 2.3 A commonly used gradation specification for soil nailing shotcrete [3]

Metric Sieve (mm)	Percentage Passing by Weight (%)
12	100
10	90-100
5	70-85
2,5	50-70
1,25	35-55
0,63	20-35
0,315	8-20
0,160	2-10

2.7.3.2 Final Facing

"Final Facing" is usually installed following completion of the excavation to final grade [3].

1. Cast-in-Place (CIP) Reinforced Concrete Facing :

The most common final facing used to date on permanent walls is cast-in-place (CIP) reinforced concrete (typically 200 mm minimum thickness). This type of facing can be readily adapted to satisfy a variety of aesthetic and durability criteria. Permanent facings consisting of CIP concrete are placed over the shotcrete following completion of the excavation to full height. Typical structural CIP reinforced concrete facing over temporary shotcrete is shown in figure 2.17. Less commonly, a second layer of shotcrete has also been used as the final facing. In addition, the shotcrete can be colored either by adding coloring agent to the mix or by applying a pigmented sealer or strain over the shotcrete surface [3].

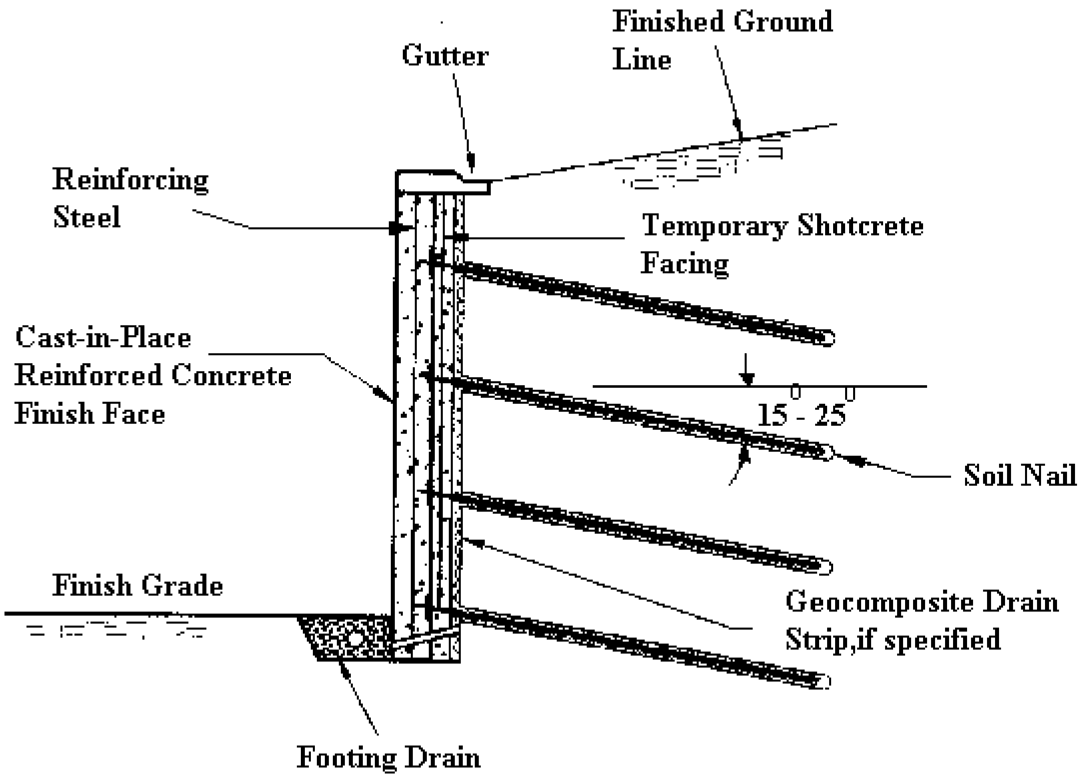


Figure 2.17 Typical structural Cast-in-Place reinforced concrete facing over temporary shotcrete [3]

2. Precast Concrete Facing :

Precast concrete facing panels have also been used as final facings, and can be attached to the construction facing in a variety of ways. The precast panels can consist of smaller modular units or of full-height tilt-up panels (Figure 2.18). One disadvantages of smaller modular system is the difficulty of providing adequate long-term corrosion protection to all the attachment devices. A further disadvantages of the smaller modular panels is the difficulty to attaching the panels to the nail heads. A disadvantage of the full-height precast panels is that they are practically limited to wall heights of about 8 m because of weight and handling limitations. Galvanized welded wire mesh has also been used as a final facing with cemented materials. Typical architectural precast concrete panel finish face is shown in figure 2.19.

Figure 2.18 Examples for precast concrete panel finish face [27,28]

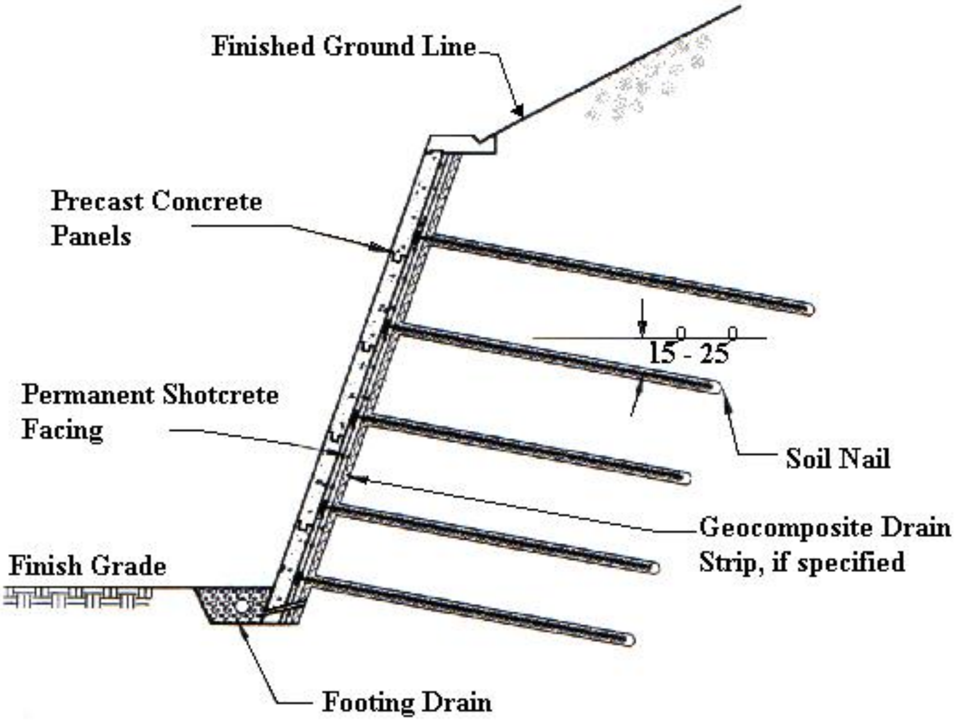


Figure 2.19 Typical architectural precast concrete panel finish face [3]

2.8 CONSTRUCTION METHODS

1. Excavation :

Before excavation, it is necessary to ensure that all surface water will be controlled during the construction process. The initial cut is excavated to a depth slightly below the first row of nails, typically 1 to 2 m depending on the ability of the soil to stand unsupported for a minimum period of 24 to 48 hours. Where face stability is problematical for these periods of time, a stabilizing berm can be left in place until the nail has been installed and final trimming then takes place just prior to application of the facing. Another method of dealing with face stability problems includes placing of a flash coat of shotcrete. It is generally the case that face stability problems are likely to be most severe during the first one or two excavation stages, because of the presence of near-surface weathered and weakened materials or, in urban environments, the presence of loose fills or voids often associated with buried utilities [2,3].

Mass excavation is done with conventional earth moving equipment. Final trimming of the excavation face is typically done with a backhoe or hydraulic excavator. Ground disturbance during excavation should be minimized and loosened areas of the face removed before shotcrete facing support is applied. The excavation face profile should be reasonably smooth and regular in order to minimize subsequent quantities.

A level working bench on the order of 10 m width is typically left in place to accommodate the drilling equipment used for nail installation.

2. Nail Hole Drilling and Drilling Methods :

Nail holes are drilled at predetermined locations (Figure 2.20) to a specified length and inclination using a drilling method (Figure 2.21) appropriate for the ground. Typical nail spacings are 1 to 2 m both vertically and horizontally. Typical nail lengths are 70 to 100 percent of the wall height and nail inclinations are generally on the order of 15 degreed below horizontal to facilitate grouting [2,3].

Drilling methods include both open hole methods (rotary or rotary percussive methods using air flush, and dry auger methods) and cased hole methods for less stable ground (single tube and duplex rotary methods with air or water flush, and hollow stem auger methods) [2,3]. Typical drilling equipment and methods are summarized in table 2.4.

The method of drilling depends on the site and ground conditions, but is most frequently “open hole” drilling. Open hole drilling is used to install about 80 to 90 percent of all soil nails. Augering is the method most commonly used to construct open holes, with diameters ranging from 100 mm to 300 mm. The most common grouting method used with open hole drilling is the low pressure tremie method. The nail grout is subsequently introduced to the drillhole using a tremie pipe to place the grout from the bottom to the top of the drillhole as the pipe is slowly withdrawn [3].

Another less common open hole drilling method is the rotary-percussive method, which displaces soil by drilling and driving drill rods.

Cased hole methods of drilling may be required in more difficult ground and are used to install only an estimated 10 to 20 percent of drilled-in soil nails. Cased hole methods of drilling include the single tube and flushing the cuttings outside the tube with air, water or a combination of water and air. The “duplex” rotary method is another cased hole method sometimes used, and is similar to the single tube rotary method, except that it uses both an inner and outer casing, which allows drill cuttings to be removed through the annular space between the inner and outer casing. Cased drill hole sizes are generally 90 mm to 150 mm in diameter. Hollow-stem augers, with grout pumped through the auger stem as the auger is withdrawn, is another cased method [2,3].

Table 2.4 Drilling methods and procedures [3]

Drill Rig Type	Drilling Method	Open Hole	Cased or Auger-Cast	Drillhole Diameter (mm)	Drill Bit Types	Cutting Removal	Comments
Auger	Lead Flight Kelly-bar Driven	Yes	No	100-300	Rock, Soil, Drag, etc.	Mechanical	Hydraulic rotary auger methods for drilling competent soils or weathered rock. Predominant drilling method for soil nail installation work.
	Sectional Solid Stem	Yes	No				
	Sectional Hollow Stem	Yes	Yes			Mechanical (air support)	
	Continuous Flight Solid Stem	Yes	No			Mechanical	
	Continuous Flight Hollow Stem	Yes	Yes			Mechanical (air support)	
Rotary	Single-Stem Air Rotary	Yes	No	100-200	Button, Roller, Drag, etc.	Compressed Air	Hydraulic rotary methods for drilling competent soils,rock, or mixed ground conditions.
	Duplex Air Rotary	Yes	Yes				
	Sectional Solid Stem Auger	Yes	No	100-300	Rock, Soil, Drag, etc.	Mechanical	Hydraulic rotary auger methods for drilling competent soils or weathered rock.
	Sectional Hollow-Stem Auger	Yes	Yes			Mechanical (air support)	
Air Track	Single- Stem Air Rotary	Yes	No	100-300	Button, Roller, Drag, etc.	Compressed Air	Pneumatic rotary methods for drilling non-caving competent soils or rock.

Figure 2.20 X marks the spot where the soil nails are to be inserted [29]

Figure 2.21 Rotary-bit drill, typically used for drilling into the soil prior to installation of nails [29]

3. Nail Installation and Grouting :

To minimize the chances of hole caving, open hole tremie grouting should take place as soon as practical after drilling and tendon insertion.

Grouting takes place under gravity or low pressure from the bottom of the hole upwards, either through a tremie pipe for open-hole installation methods or through the drill string (or hollow stem) or tremie pipe for cased installation methods.

Grout should be injected by tremie pipe inserted to the bottom of the drillhole, so that the grout evenly and completely fills the hole from the bottom to the surface, and without air voids. The grout should flow continuously as the tremie pipe is withdrawn. The withdrawal rate should be controlled to ensure that the end of the tremie pipe is always below the grout surface [3].

Plastic centralizers are commonly used to center the nail in the drillhole. However, where the nails are installed through a hollow stem auger, centralizers are generally ineffective and a stiffer (200 mm or lower slump) grout mix is used to maintain the position of the nail and prevent it from sinking to the bottom of the hole. The nails, which are commonly 19 to 35 mm bars (yield strength in range of 420 to 500 N/mm^2), are inserted into the hole and the drillhole is filled with cement grout to bond the nail bar to the surrounding soil. However, nail sizes smaller than 25 mm can cause installation problems for moderate- long nail lengths due to their low stiffness.

4. Placing Drainage System :

A 400 mm wide prefabricated synthetic drainage mat, placed in vertical strips between the nail heads on a horizontal spacing equal to that of the nails (Figure 2.22), is commonly installed against the excavation face before shotcreting occurs, to provide drainage behind the shotcrete face. The drainage strips are extended down to the base of the wall with each excavation lift and connected either directly to a footing drain or to weep holes that penetrate the final wall facing. These drainage strips are intended to control seepage from perched water or from limited surface infiltration following construction. If water is encountered during construction, short horizontal drains are generally required to intercept the water before it reaches the face [3].

Figure 2.22 Installation of drainage strips along one construction layer of the soil nail wall [29]

5. Placing Construction Facing and Installing Bearing Plates :

The construction facing typically consists of a mesh-reinforced wet-mix shotcrete layer (Figure 2.23) on the order of 100 mm thick, although the thickness and reinforcing details will depend on the specific design. Following placement of the shotcrete, a steel bearing plate (typically 200 mm to 250 mm square and 19 mm thick) and securing nut are placed at each nail head and the nut is hand wrench tightened sufficiently to embed the plate a small distance into the still plastic shotcrete [3].

Figure 2.23 The construction facing consists of a mesh-reinforced wet-mix shotcrete layer [23]

6. Placing Final Facing :

For architectural and long term structural durability reasons, a CIP concrete facing is the common final facing. The CIP facing is typically structurally attached to the nail heads by the use of headed studs welded onto the bearing plates. Under appropriate circumstances, the final facing may also consist of a second layer of structural shotcrete applied following completion of the final excavation. Pre-cast concrete panels may also be used as the first facing for soil nail walls [3]. The completed soil nail wall is shown in figure 2.24.

Figure 2.24 The completed soil nail wall [30]

2.9 APPLICATION OF SOIL NAIL WALL

Soil nail walls have been found to be an economical solution to many soil reinforcement and excavation support problems. The following section lists some of the typical applications for soil nail walls and some of their benefits [20].

1. Alternative to Tieback Wall for Temporary or Permanent Excavation Support:
   - Eliminates the time and expense of placing H-piles.
   - Eliminates labor associated with placing timber lagging or sheet piling.
   - Eliminates the need for expensive structural facing systems.
   - By placing a structural face on a soil nail wall, it can be used as the permanent foundation wall, saving the time and money associated with an additional construction step.
   - Decreases right-of-way requirements, since the length required for soil nails is shorter than that for tiebacks.
2. Alternative to Cast in Place Walls (CIP) in Cuts:

Cast-in-place walls in cuts will require temporary shoring and over excavation to be able to install wall footings. A soil nail wall requires no shoring and can use a smaller footing (Figure 2.25).

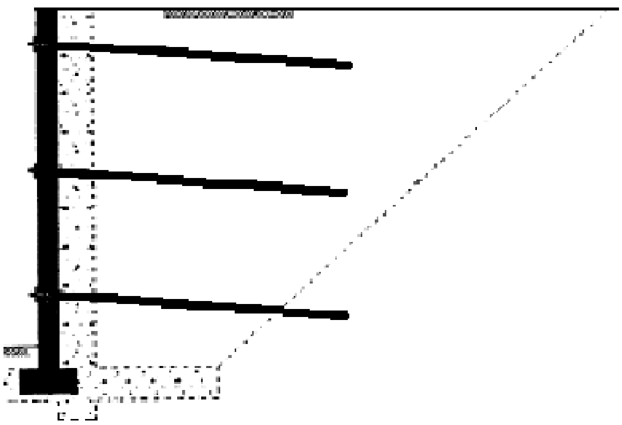


Figure 2.25 Soil Nail Wall System Replacing Cast-in-Place Wall [20]

3. Repair and Reconstruction of Existing Retaining Wall Systems:

Replacement and reconstruction of a failed timber or concrete crib wall, MSE wall, gabion wall, or CIP wall is very expensive. An alternative is to reinforce the failed wall with soil nails and replace or repair the facing. This eliminates a very expensive construction step of excavating the failed wall, especially if the wall is supporting another structure (Figure 2.26).

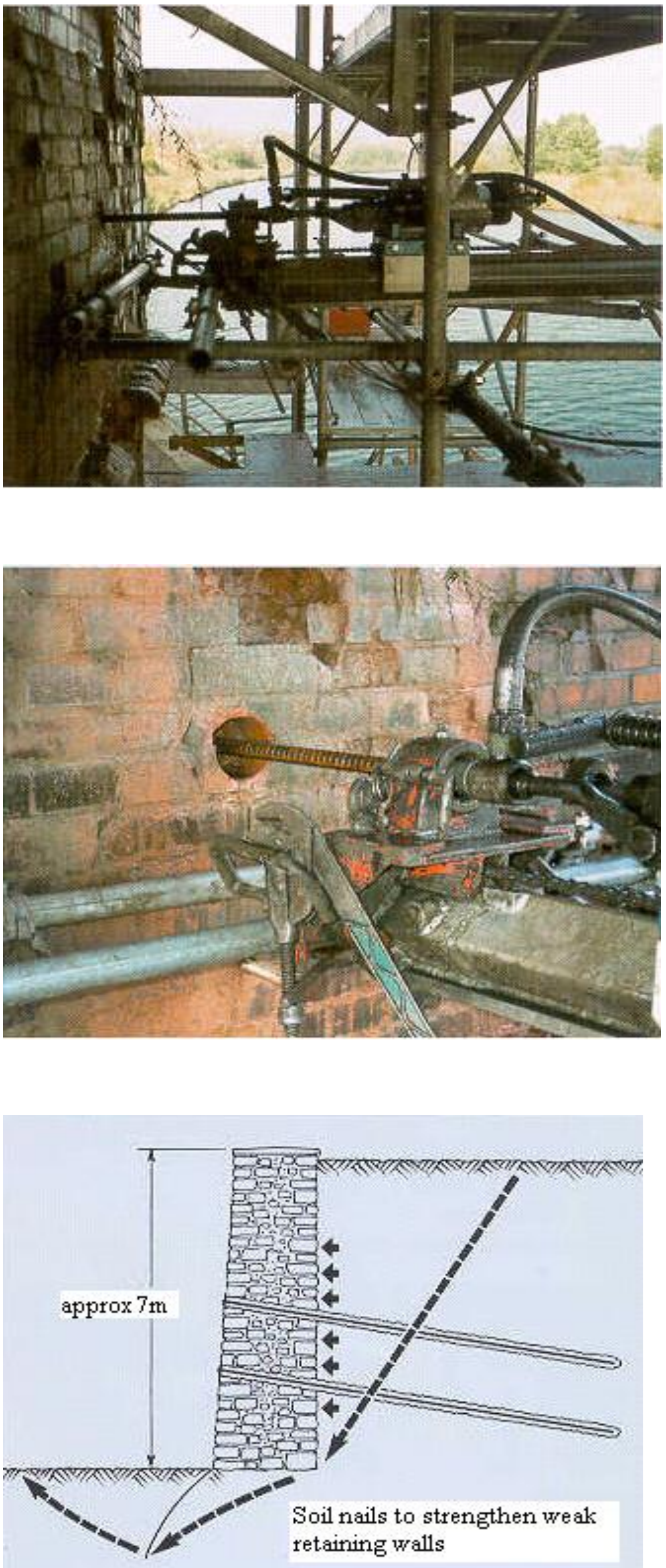


Figure 2.26 Repair of existing retaining wall system [31]

4. Roadway Widening under Existing Bridges:

Soil nail walls can eliminate construction steps associated with temporary and permanent walls needed for widening roadways adjacent to existing highway bridges. Soil nail walls can be combined with permanent facings, thus providing a permanent wall for support of bridge fills without the need for temporary shoring by using top down construction sequence (Figure 2.27).

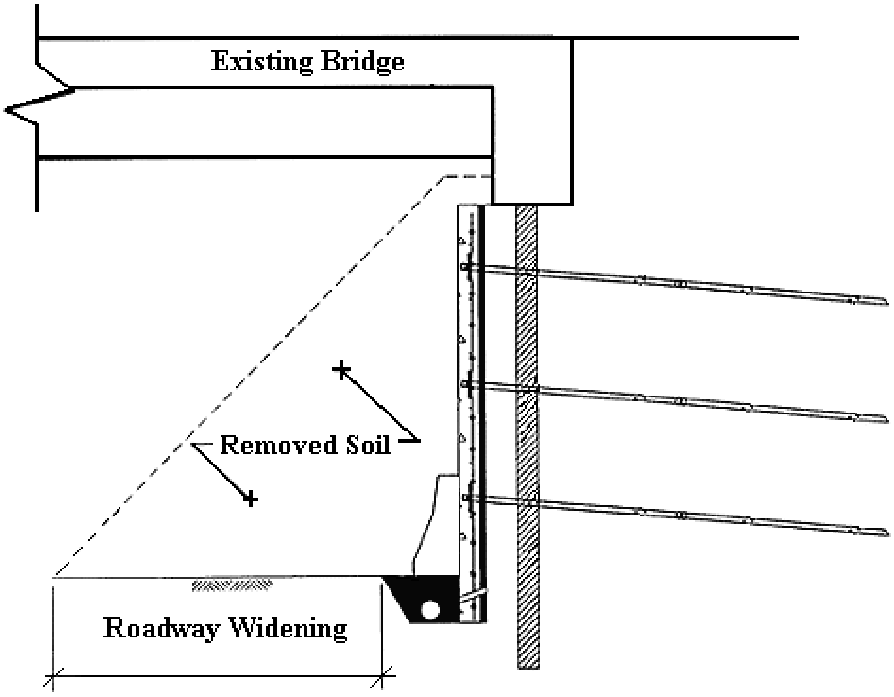


Figure 2.27 Soil nail wall system used for roadway widening at bridge abutment [20]

5. Landslide Remediation:

Soil nail walls can be used to reinforce failed slopes and walls in-situ. Soil nails must be drilled beyond the failure surface to a depth great enough to mobilize the nail tensile strength. This analysis is similar to the design of a reinforced fill slope, however, soil nails enable this remediation to be performed in-situ without removal and replacement (Figure 2.28 and 2.29).

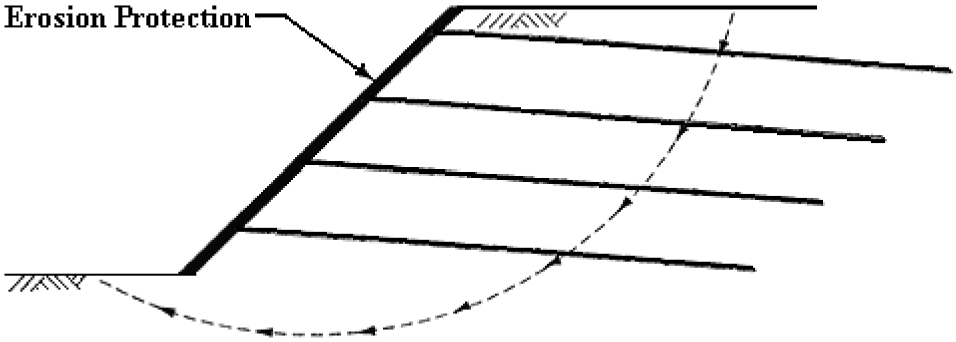


Figure 2.28 Soil nail wall system for landslide remediation [20]

Figure 2.29 Slip circle failure occur due to flowing water, trapped water, added overburden or erosion at the base of the slope [19]

2.10 BEHAVIOR OF SOIL NAIL WALLS

2.10.1 Fundamental Mechanism of Soil Nail Walls

The fundamental mechanism of Soil Nail Retaining Structures is the development of tensile forces in the “passive” reinforcements. In the case of a soil nail wall constructed from the top-down, the lateral expansion of the reinforced zone is associated with removal of lateral support as excavation proceeds following installation of each level of reinforcement [3].

Loads are developed within the soil nails primarily as a result of the frictional interaction between the nail and the soil, and secondarily by the soil-structure interaction between the facing and the soil. The latter phenomenon is responsible for the development of tensile load at the head of the nail, and the nail head load is typically some fraction of the maximum nail load. The maximum tensile load within each nail occurs within the body of the reinforced soil at a distance from the facing that depends on the vertical location of the nail within the wall. The line of maximum tension within the nails is often considered as dividing the soil mass into two separate zones [3].

a. an “active zone” close to the facing, where the shear stresses exerted by the soil on the reinforcement are directed outward and tend to pull the reinforcement out of the ground.
b. a “resistant zone”, where the shear stresses are directed inward and tend to restrain the reinforcements from the pull-out.

This behavior is shown on figure 2.30. It should be noted that the line of maximum tension does not correspond to the conventional critical slip surface.

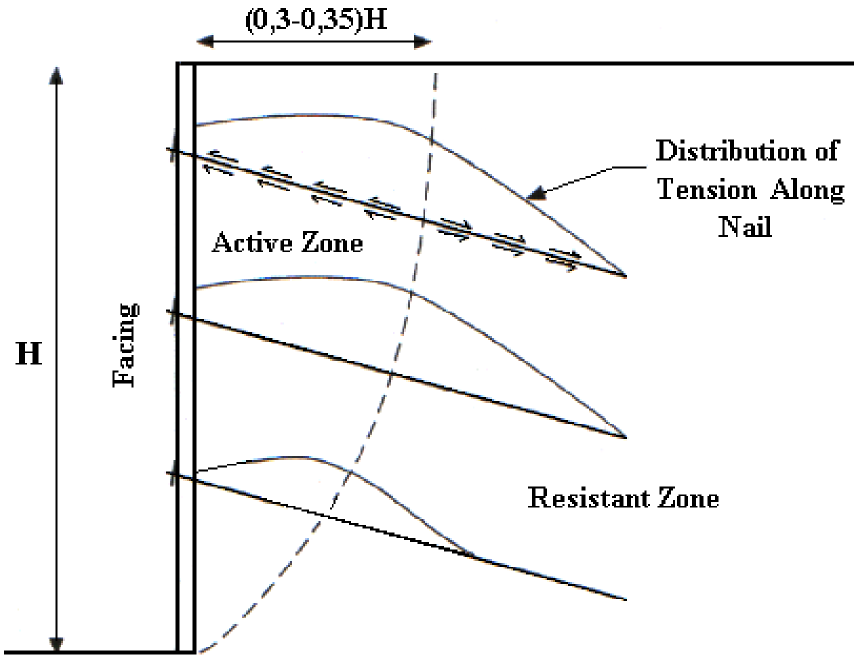


Figure 2.30 Soil nail behavior [3]

The reinforcement acts to tie the active zone to the resistant zone. For stability to be achieved, the nail tensile strength must be adequate to provide the support force to stabilize the active block. The nails must have a sufficient length into the resistant zone to prevent a pull-out failure. In addition, the combined effect of the nail head strength and the pullout resistance of the length of the nail between the face and the slip surface must be adequate to provide the required nail tension at the slip surface.

2.10.2 Types of Failure of Soil Nailed Walls

The potential failure surfaces can be located inside or outside the soil nailed retaining structures (Figure 2.31).

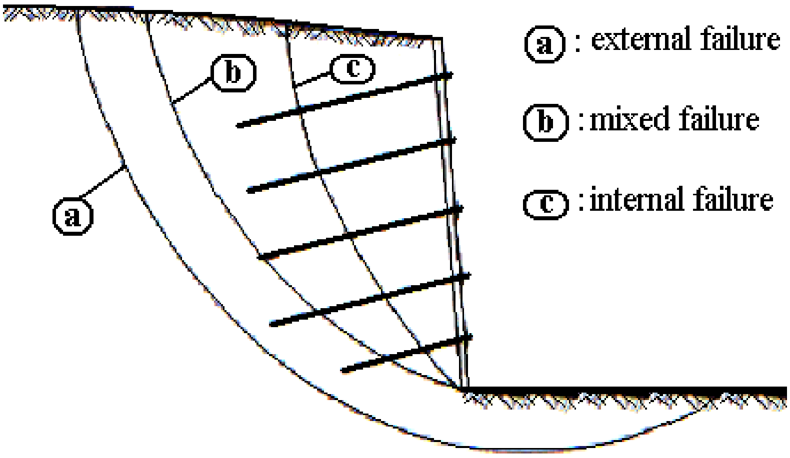


Figure 2.31 Different types of failure to be analyzed [4]

2.10.2.1 Failure by Breakage of the Nails (Internal Failure)

The failure surface that develops in the soil is very close to the line of maximum tension, which can, therefore, be considered as a potential failure surface (Figure 2.32).

With flexible nails, failure is sudden and without warning. The resistance to bending of the nails allows greater deformations before failure; this forms a warning sign and allows more progressive failure to take place [4].

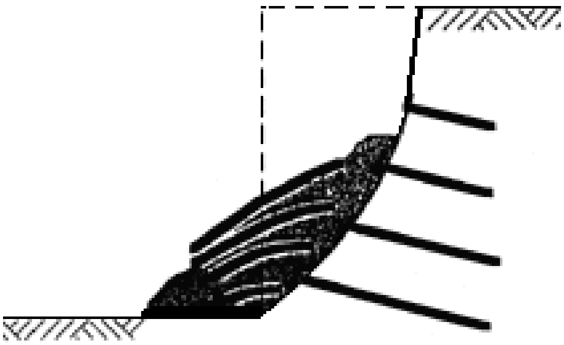


Figure 2.32 Failure by breakage of the nails [20]

This type of failure can occur in the cases listed below [4]:

1. It may come from under designing the cross sections of nails.
2. It may be induced by corrosion of the steel bars in the nails.
3. It may be produced by a surcharge on top of the wall, if the wall has not been designed to resist it.
4. It may be induced by saturation of the wall under the effects of water infiltrations (rain or thaw).
5. It may be caused by the ice lenses in frost-susceptible soils.

2.10.2.2 Failure by lack of adherence (Internal Failure):

The failure by lack of adherence is characterized by the fact that the nails do not have sufficient length in the passive zone to be able to balance the maximum tensions (Figure 2.33). The nails are then pulled out of the soil. This type of failure is not usually sudden, except in some cases during construction, and that large deformations develop [4].

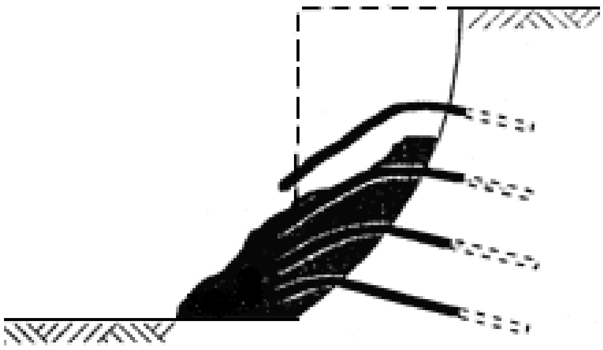


Figure 2.33 Failure by lack of adherence [20]

This type of failure can occur [4]:

1. In fine-grained soils under the effect of saturation or increase in moisture content.
2. During construction, if the length of the nails at the head of the wall is insufficient.

2.10.2.3 Failure due to excessive height of continuous excavation (Internal Failure):

During the construction of a wall, if the height of the excavation phase is too great, fairly sudden failure can occur. In this type of failure, the soil flows behind the facing due to successive elimination of the arch effects [4].

The nails deform through bending but may not break (Figure 2.34).

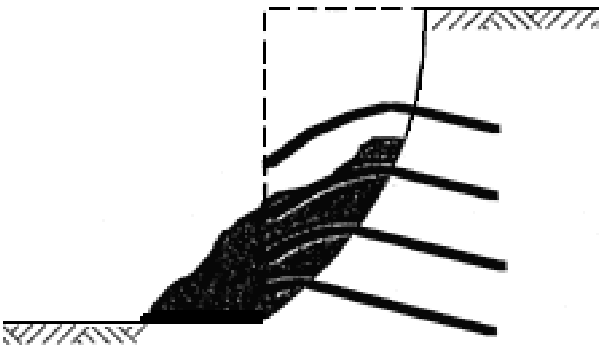


Figure 2.34 Failure due to excessive height of continuous excavation[20]

To prevent this failure, the excavation height must be kept lower than the critical height.

2.10.2.4 External failure and mixed failure

The external failure of a soil nailed wall occurs generally by sliding along a failure surface, affecting the whole structure and going through the foundations [4].

This type of failure is common to all retaining structures. External failure is due to either poor quality foundation soils or to insufficient length of the nails resulting in global failure that, in part, takes the form of sliding of the wall on its base (Figure 2.35).

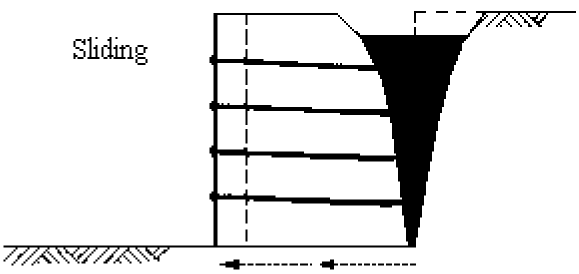


Figure 2.35 Sliding of the wall on its base (external failure) [20]

Mixed failure relates to a failure surface both in the wall and outside the wall (Figure 2.31). It combines both internal instability and external instability of the wall. Mixed failure is generally due to nails being of insufficient length, associated with a defect in strength of the nails or in the unit skin friction [4].

2.10.3 Distribution of Nail Forces

Figure 2.30 shows a typical distribution of nail forces for a soil nail retaining wall with a horizontal backslope. For a near-vertical wall with a horizontal backslope, the line of maximum tension within the reinforced zone is typically curvilinear and intercepts the surface at about 0,3H to 0,35H back from the wall. Considering the nail lengths are typically on the order of 0,6H to 0,8H, this implies that in the upper part of the reinforced zone, the maximum nail force tends to occur at about the mid-length of the nail. In the lower portions of the reinforced zone, the point of maximum tension moves closer to the wall face. The nail tension at the face is generally less than the maximum nail tension. The nail tensions are developed gradually as the excavation proceeds following nail installation [1,4].

2.10.4 Deformation Behavior

During construction of a soil nail wall from the top-down, the reinforced soil zone tends to rotate outwards about the toe of the wall as part of the process of mobilizing tensile loads within the nails. Hence, maximum horizontal movements occur at the top of the wall and decrease progressively towards the toe of the wall. This is due to the influence of the L/H ratio, which decreases as the wall is being built. At the top of the wall, three displacements can be defined δ_h , δ_v ,δ_0 (Figure 2.36).

On the ground surface at the top of a soil nailed wall, the lateral and vertical displacements which are maximum at the edge of the wall decrease to zero over a length, λ, which is function of the soil type (coefficient κ), the inclination of the wall (η), and the wall height (H) according to the empirical formula $\lambda = H\,(1\text{-}\tan\,\eta)\,\kappa$. Based on the empirical results, which are summarized in table 2.5, one can estimate a priori the amount of differential settlements and extension the foundations of an existing building near a soil nailed wall will have to undergo [1,4,6].

The horizontal displacement δ_h at the head of the facing is about equal to the vertical displacement δ_v.

Displacement δ_0 is generally comprised between 4H/10 000 and 5H/10 000; its value varies inversely to the L/H ratio and also depends on the nature of the soil.

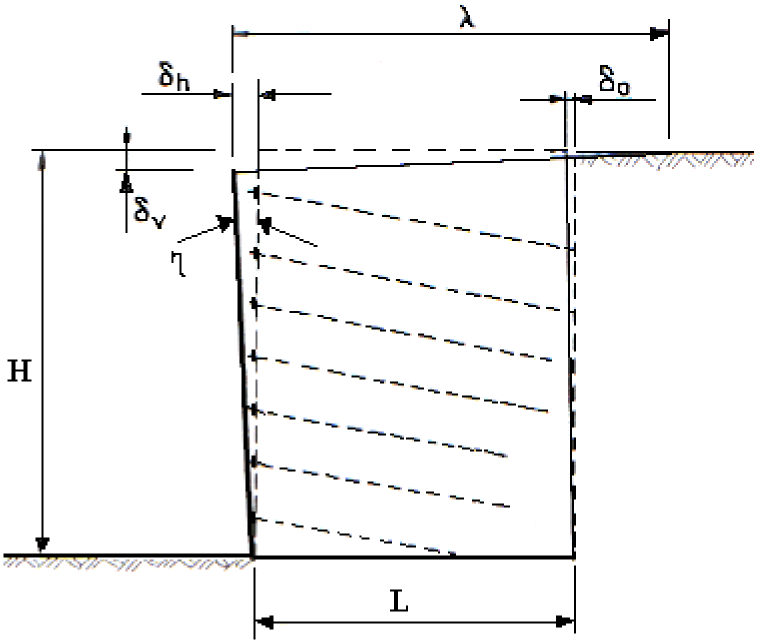


Figure 2.36 Definitions of displacements [4]

Table 2.5 Summary of data on displacements [4]

Type of Soil	Weathered Rocks Stiff Soils	Sandy Soils	Clayey Soils
$\delta_v = \delta_h$	H / 1000	2H / 1000	4H / 1000
Coefficient κ	0,8	1,25	1,5

$$\lambda = H\,(1 - \tan\eta)\,\kappa$$

3.IN-SITU INVESTIGATION AND TESTING

3.1 SITE INVESTIGATION

To construct a soil nailed wall on a project depends on the existing topography, subsurface conditions, soil/rock properties, and the location and condition of adjacent structures. It is, therefore, necessary to perform a comprehensive site investigation to evaluate site stability, adjacent structure settlement potential, drainage requirements, underground utilities and groundwater, before designing a soil nailed wall [1,3].

Subsurface investigations must explore not only the location of the face of the soil nailed structure, but the region of the anticipated bond length of the nail. Each project must be treated separately, as both the soil conditions and risks may vary widely. A well-planned site investigation should include a review of the regional geology, a field reconnaissance, a subsurface exploration and laboratory testing [3]. The site investigation should provide adequate information to design a stable soil nailed system.

1. Regional Geology:

A review of the regional geology should be performed prior to conducting a field reconnaissance or subsurface exploration to better understand the geology and groundwater conditions of the region. The information acquired in this first phase of the site evaluation will be used to further develop the field reconnaissance and subsurface exploration. Information concerning the regional geology may be obtained from geologic maps, air photographs, surveys and soils reports for adjacent or nearby sites.

2. Field Reconnaissance:

A well planned and conducted field reconnaissance should consist of collecting any existing data relating to the subsurface conditions and making a field visit to [20]:

- Select limits and intervals for topographic cross-sections.

- Observe surface drainage patterns, seepage and vegetative characteristics to estimate drainage requirements. Corrosion of existing drainage structures should be noted to identify if a corrosive environment may exist for shotcrete and/or steel materials.
- Study surface geologic features including rock outcroppings and landforms. Existing cuts or excavations should be used to identify subsurface stratification.
- Determine the extent, nature, and situation of any above or below ground utilities, basements and/or substructures of adjacent structures which may impact explorations or construction.
- Assess available right-of-way.
- Determine areas of potential instability, such as deep deposits of weak cohesive and organic soils, slide debris, high groundwater table, bedrock outcrops, etc.

3. Subsurface Exploration

The subsurface exploration program may consist of soil borings, test pits, cone penetration tests, soil soundings, etc. The number, type, and location of the subsurface explorations are usually determined by the geotechnical engineer, based on the results of the field reconnaissance. The exploration must be sufficient to evaluate the geologic and subsurface profile in the area of construction. The following minimum guidelines are suggested for the subsurface exploration for a soil nailed wall [20]:

- Soil borings should be performed at intervals of 30 m along the alignment of the soil nailed wall face and 45 m along the back of the reinforced soil structure (Figure 3.1). The width of the soil nailed structure may be assumed as 1.0 to 1,5 times the height of the wall. For sloping ground conditions behind the wall face, the width of the soil nailed structure may be assumed to be 1.5 to 2,0 times the wall height.
- The boring depth should be controlled by the general subsurface conditions. In areas of where rock is not encountered, the boring should extend at least to a depth equal to twice the height of the earth structure. Where bedrock is encountered at a reasonable depth, rock cores should be obtained for a length of approximately 3 m. This coring will be useful in distinguishing between solid rock and boulders.

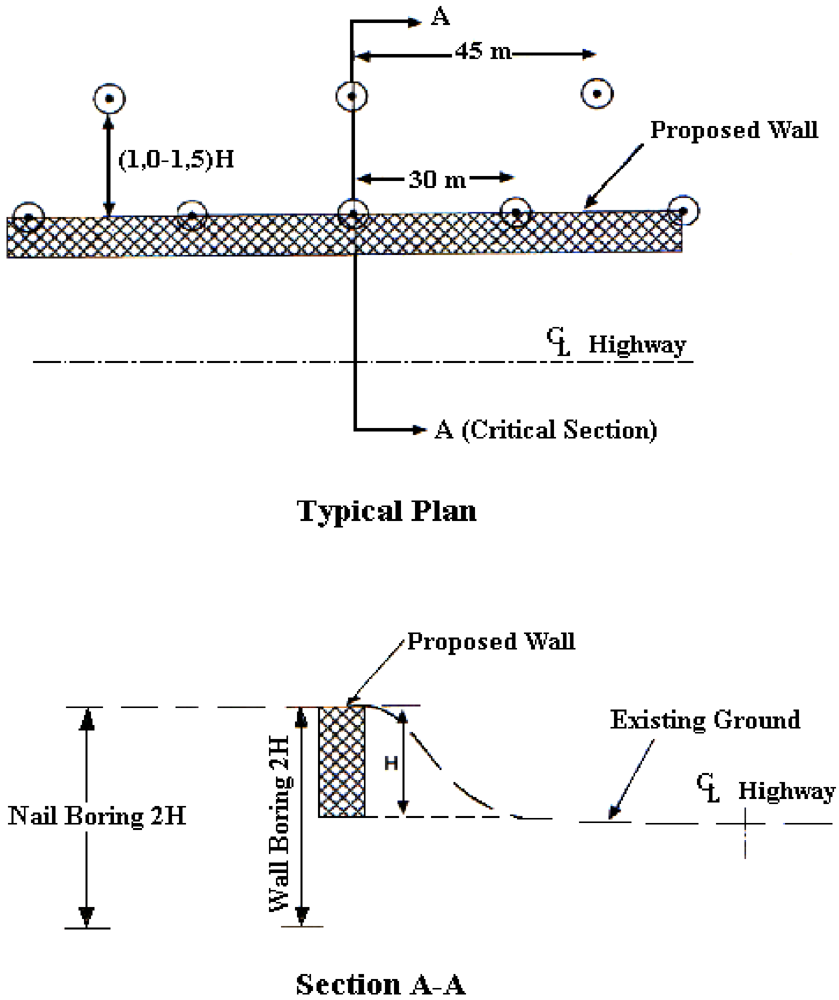


Figure 3.1 Site exploration guideline for soil nail walls [3]

- In each boring, soil samples should be obtained at 1,5 m intervals and at changes in strata for visual identification, classification, and laboratory testing. In each boring, careful observation should be made for the prevailing groundwater table, which should be observed at the time of sampling but also at later times to obtain an understanding of the change in groundwater table with time.
- Additional information from in-situ testing such as dilatometer, and pressuremeter may be conducted to provide soil modulus values.
- Obtain bulk samples of the subsurface soils to be used in the laboratory testing program.
- Test-pit explorations should be performed to help assess whether or not the excavated face will stand while temporarily unsupported during the stage of excavation prior to shotcreting the face.

4. Laboratory Testing:

Soil samples should be visually examined and appropriate tests performed for classification according to the Unified Soil Classification System. These tests will permit the engineer to decide what further tests will best describe the engineering behavior of the soil at a given project site. Index testing includes determining the moisture content, Atterberg limits, compressive strength and gradation [20].

Shear strength determination from unconfined compression tests, direct shear tests, or triaxial compression tests will be needed for the stability analysis. Both undrained and drained (effective stress) strength parameters will be needed for cohesive soils to permit evaluation of both long-term and short-term conditions.

Properties to indicate the potential aggressiveness of the in-situ soil within the reinforced zone should be measured. The tests include: pH, electrical resistivity, and salt content (sulfate, sulfides, and chlorides). These test results will provide necessary information for planning degradation potential and protection [3,20].

5. Final Feasibility Evaluation [3]

Based on the results of the subsurface exploration and subsequent laboratory testing program, a final feasibility evaluation can be made to determine if a successful soil nail design can be implemented with a relatively high degree of confidence. This requires an understanding of ground conditions for which soil nailing is well suited and the ground conditions that are problematic [3].

3.2 ESTIMATING SOIL/NAIL INTERACTION

Two types of interaction develop in nailing used in retaining structures [3,4,5]:

1. The most important interaction is the shear stress (skin friction) applied by the soil along the nail, which induces tension in the nails.
2. A second, less important interaction is the passive pressure of the earth along the nail during the displacement of the latter. The passive earth pressure mobilized makes possible the bending moment and shear force mobilized in the nails, this mobilization occurs only if a shear zone develops in the soil nailed mass.

The nail pullout resistance can be affected by [3]:

- Soil or rock type and shear strength.
- Roughness of drillhole wall (will vary with drilling method used).

- Final drillhole diameter.
- Loose drill cuttings left along the bottom of the drillhole.
- Contractor drilling and grouting techniques and workmanship.
- Amount of time hole left open before grouting.

A. Cohesionless (Granular) Soil:

For tremie or low pressure grouted nails in dry cohesionless soils, data reported in the literature suggest the following ranges of ultimate friction limit (table 3.1).

Table 3.1 Estimated pull-out resistance in cohesionless soils [3]

Construction Method	Soil Type	Ultimate Friction Limit (kN/m^2)
Open Hole	Non-Plastic Silt	20-30
	Medium dense sand and silty sand	50-75
	Dense silty sand and gravel	80-100
	Very dense silty sand and gravel	120-240
	Loess	25-75

B. Cohesive Soil:

Typical values of ultimate friction limit for cohesive soils are indicated in table 3.2.

Table 3.2 Estimated pull-out resistance in cohesive soils [3]

Construction Method	Soil Type	Ultimate Friction Limit (kN/m^2)
Open Hole	Stiff Clay	40-60
	Stiff Clayey Silt	40-100
	Stiff Sandy Clay	100-200

C. Rock:

Estimated ultimate pullout resistance for different rock types are given in table 3.3.

Table 3.3 Estimated pull-out resistance in rock [3]

Construction Method	Rock Type	Ultimate Friction Limit (kN/m^2)
Rotary Drilled	Marl/Limestone	300-400
	Phillite	100-300
	Chalk	500-600
	Soft Dolomite	400-600
	Fissured Dolomite	600-1000
	Weathered Sandstone	200-300
	Weathered Shale	100-150
	Weathered Schist	100-175
	Basalt	500-600

In the design procedure, the nail pullout resistance is expressed in terms of force per unit length of nail, kN/m.

3.3 GROUND CONDITIONS BEST SUITED FOR SOIL NAILING

In general, the economical use of soil nailing requires that the ground be able to stand unsupported in a vertical or steeply-sloped cut of 1 to 2 m in height for one to two days. In addition, it is highly desirable that an open drill hole can maintain its stability for at least several hours. The following ground types are considered suitable for soil nailing [3];

1. Residual soils and weathered rock without unfavorably oriented, low strength structure.
2. Stiff cohesive soils such as clayey silts and low plasticity clays that are not prone to creep.
3. Naturally cemented or dense sands and gravels with some cohesion.
4. Fine to medium homogeneous sands with capillary cohesions of at least 5 kN/m^2 associated with a natural moisture content of at least 5%. This soil type can sometimes exhibit face stability problems when south facing slopes are subject to drying by the sun.
5. Above the ground water table.

3.4 GROUND CONDITIONS NOT WELL SUITED FOR SOIL NAILING

The ground types not considered well suited to soil nailing or limit its application is given in table 3.4.

Table 3.4 The ground types not considered well suited to soil nailing or limit its application [3]

Types of Soils Not Well Suited for Soil Nailing	The Problems Caused by These Types of Soils
1) Loose clean granular soils with field standard penetration N values lower than about 10 or relative densities of less than about 30%.	These types of soils will not generally exhibit adequate stand-up time and are also sensitive to vibrations induced by construction equipment.
2) Granular cohesionless soils with a uniformity coefficient of less than 2, unless in a very dense conditions.	During construction these types of soils will tend to ravel when exposed due to a lack of apparent cohesion.
3) Soils containing excessive moisture or wet pockets.	They tend to slough and create face stability problems when exposed i.e., the apparent cohesion is destroyed.
4) Organic soil or clay soils with a liquidity index greater than 0,2 and undrained shear strength less than 50 kN/m^2.	They may continue to creep significantly over the long term and may also exhibit a significant decrease in the soil-grout adhesion and nail pullout resistance.
5) Highly frost-susceptible and expansive(swelling) soils.	These soils can result in significant increases in the nail loading near the face; wall damage has been reported under these conditions.
6) Highly fractured rocks with voids and open graded coarse granular materials	These types of soils require special care because of the difficulty of satisfactorily grouting the nails.

4.DESIGN OF SOIL NAILED RETAINING STRUCTURES

4.1 INTRODUCTION

The scientific studies began in Germany in 1975 with several series of model tests and seven large-scale tests. Bearing and failure mechanisms were observed, earth pressures were measured, the influence of nail length and spacing was examined and the internal and external stability of the “gravity wall” were studied both in non-cohesive and cohesive soils. Based on these observations and measurement results, calculation and design methods have been developed [8]. The soil nailing design parameters are shown in figure 4.1.

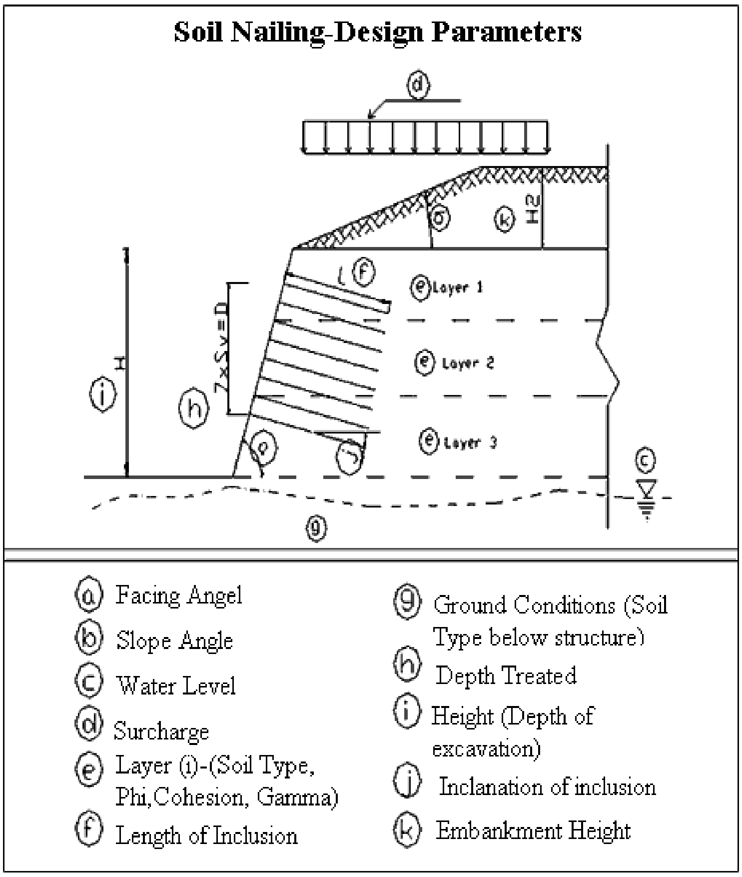


Figure 4.1 Design parameters [27]

The most important results were [8];

1. The nailed soil structure behaves like a gravity wall.
2. The required nail length for the general case of a vertical wall face and a horizontal ground surface lies in the range of 0,5 to 0,8 times the height of the wall.
3. The spacing of the nails should be less than 1,5 m, i.e. the reinforcement ratio should be at least one nail per 2,25 m^2.
4. The earth pressure onto the wall face may be assumed with uniform rectangular distribution. Its magnitude is on the order of 0,4 to 0,7 times the active Coulomb's earth pressure.
5. No negative effects on the stability or the deformation of the wall due to dynamic loading were found.

The design methods that have been most commonly used in Europe (the French and German methods) and the United States (the Davis method) consider only a global stability analysis of the structure and the retained ground. They involve different assumptions with regard to the shape of the failure surface, the mode of soil-reinforcement interaction and type of resisting forces generated in the nails [8].

4.2 DESIGN METHODS FOR SOIL NAILED RETAINING STRUCTURES

The fundamental concept of soil nailing consists of placing in the ground passive inclusions, closely spaced, to restrain displacements and limit decompression during and after excavation [8].

The design procedure for a soil-nailed retaining structures should include the following steps [5]:

1. For the specified structure geometry (depth and cut slope inclination), ground profile, and boundary (surcharge loadings), estimate working nail forces and location of the potential sliding surface.
2. Select the reinforcement type and verify local stability at each reinforcement level, that is, verify that nail resistance (strength and pullout capacity) is sufficient to withstand the estimated working forces with an acceptable factor of safety.
3. Verify that the global stability of the nailed-soil structure and the surrounding ground is maintained during and after excavation with an acceptable factor of safety.

4. Estimate the system of forces acting on the facing (i.e., lateral earth pressure and nail forces at the connection) and design the facing for specified architectural and durability criteria.
5. For permanent structures, select corrosion protection relevant to site conditions.
6. Select the drainage system for groundwater piezometric levels.

The available design methods for soil nailed retaining structures, can be broadly classified into two main categories [5]:

1. Limit equilibrium design methods or modified slope stability analyses, which are used to evaluate the global safety factor of the nailed structures with respect to a rotational or translational failure along potential sliding surfaces, taking into account the shearing, tension, or pull-out resistance of the inclusions crossing the potential failure surface (French Method, German Method, Davis Method and “Modified” Davis Method).
2. Working stress design methods which are used to estimate the tension and shear forces generated in the nails during construction under the design loading conditions and evaluate the local stability at each level of nails.

Table 4.1 illustrates the main features and basic design assumptions of the available design methods for soil nailed structures. The governing principles of these methods and their evaluation are briefly summarized below.

4.2.1 Limit Equilibrium Design Methods

Limit equilibrium approaches are often used in the current design of soil nailed structures under both static and seismic loads. Slope stability analysis procedures have been developed to evaluate the global stability of the soil nailed mass and/or the surrounding ground, taking into account the shearing, tension, or pull-out resistance of the inclusions crossing the potential failure surface. As in traditional slope stability analysis, limit equilibrium conditions are used to search for the most critical failure surface, which can be located either inside or outside the soil nailed retaining structure. The available design procedures involve different assumptions with regard to the shape of the failure surface, the type of soil reinforcement interaction, and the resisting forces in the inclusions. They can be broadly classified as; limit force equilibrium analysis and, multi-criteria limit equilibrium analysis [3,5].

Table 4.1 Basic assumptions of the different design approaches

FEATURES	FRENCH METHOD (Schlosser, 1983)	GERMAN METHOD (Stocker, et al., 1979)	DAVIS METHOD (Shen, et al., 1981)	“MODIFIED” DAVIS METHOD (Elias and Juran, 1988)	KINEMATICAL METHOD (Juran, et al., 1989)
Analysis	Limit moment Equilibrium Global Stability	Limit force Equilibrium Global Stability	Limit force Equilibrium Global Stability	Limit force Equilibrium Global Stability	Working stress Analysis Local Stability
Input Material Properties	Soil Parameters(c,ϕ) Limit Nail Forces Bending Stiffness	Soil Parameters(c,ϕ) Lateral Friction	Soil Parameters(c,ϕ) Limit Nail Forces Lateral Friction	Soil Parameters(c,ϕ) Limit Nail Forces Lateral Friction	Soil Parameters[$c/(\gamma H)$,ϕ] Non-dimensional bending stiffness parameter (N)
Nail Forces	Tension, Shear, Moments	Tension	Tension	Tension	Tension, Shear, Moments
Failure Surface	Circular Any input shape	Bi-linear	Parabolic	Parabolic	Log-spiral
Failure Mechanisms	Mixed	Pull-out	Mixed	Mixed	Non applicable

Soil Strength Safety Factors	1,5	1,0	1,5	1,0	1,0
Pull-out Resistance Safety Factors (FS_p)	1,5	1,5 - 2,0	1,5	2,0	2,0
Tension Bending	Yield stress Plastic Moment	Yield Stress	Yield Stress	Yield Stress	Yield stress Plastic Moment
Design	GSF	GSF	GSF	GSF	Mobilized nail forces
Soil Stratification	Yes	No	No	No	Yes
Loading	Slope Any surcharge	Slope surcharge	Uniform surcharge	Slope Uniform surcharge	Slope
Structure Geometry	Any input geometry	Inclined facing Vertical Facing	Vertical Facing	Inclined Facing Vertical Facing	Inclined Facing Vertical Facing

4.2.1.1 Limit Force Equilibrium Analysis

These methods, take into account only the tension resistance and pull-out capacity of the inclusions. As indicated in Table 4.1 the limit equilibrium analysis is conducted with specified values of factors of safety with respect to the shear strength characteristics of the soil and the ultimate soil-nail interface shear stress.

Limit force equilibrium methods were developed by several indicators. Stocker (the "German method") assumed a bilinear sliding surface. Shen, et al. ("Davis method") considered a parabolic sliding surface. Both methods take into account only the tension resistance and pull-out capacity of the inclusions [5].

Recently, two other limit force equilibrium methods have been developed in the U.S. These are the SNAIL design method and the GoldNail design method. SNAIL design method uses a bi-linear or linear failure surface. The GoldNail design method can analyze circular failure surface. Both methods consider the tensile resistance of the nails crossing the failure surface. These two methods have improvements over the other limit equilibrium methods in that they design the soil-nail-wall facing as a system and: 1) Consider the limiting pull-out capacity of the nails on both the wall and non-wall sides of the failure surface; and 2) Allow the structural face capacity of the wall facing to be incorporated into the analysis [3].

4.2.1.1.1 The German Method (Stocker et.al., 1979)

Since 1979, Stocker et al., have been proposing a limit equilibrium method for designing soil nailed walls at failure using bi-linear failure surfaces. This method, which was developed in the light of experience from laboratory tests on reduced scale models (Figure 4.2), has also been compared with tests on full-sized structures (Gassler and Gudehus, 1981) [4].

The structures' global safety factor is defined by the ratio of the sum of the available resisting limit nail forces ΣT_p (soil reaction along the failure plane, tension in the nails) with the total force ΣT (weight and loads) and then calculated using the kinematical approach of limit analysis.

$$FS = \Sigma T_p / \Sigma T \tag{4.1}$$

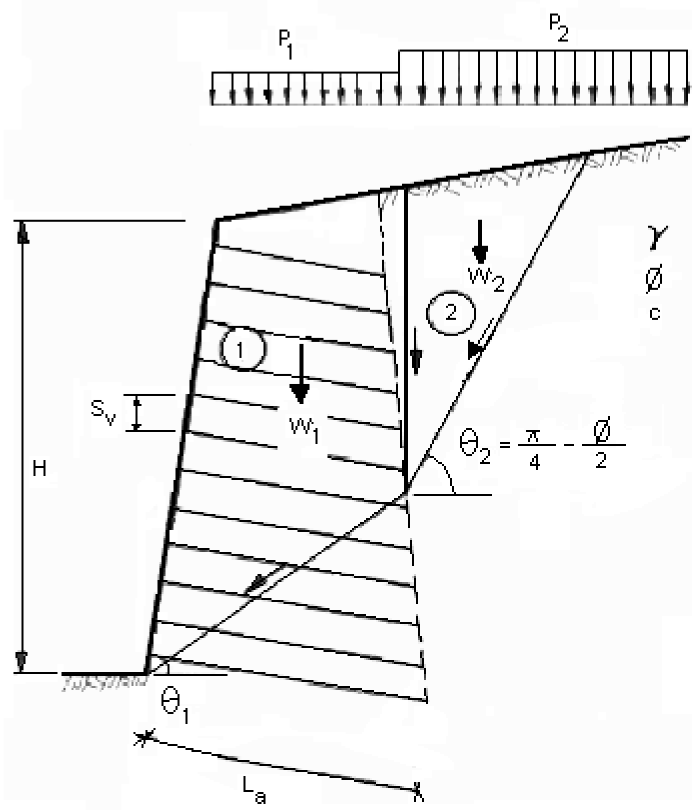


Figure 4.2 Bi-linear failure surface used in the Stocker at al., method (1979) [4]

The total force ΣT, required to maintain the limit equilibrium is readily obtained by considering the polygon of forces acting on a rigid soil wedge limited by the potential failure surface. The resisting forces T_p are provided by the pull-out capacity of the nails (pull-out capacity beyond failure surface) [1].

$$T_p = F_1 \pi D L_a \tag{4.2}$$

The inclination θ_1 of the wedge passing through the base of the wall is determined iteratively to obtain a minimum safety factor. The value of angle θ_2 is taken to be equal to $\pi/4 - \phi/2$, where ϕ is the internal friction angle of the ground [4].

The calculation is made by considering the equilibrium of two blocks sliding in relation to each other following a vertical straight line. The soil is assumed to be homogeneous and without water. If layers of soil with varying mechanical characteristics are present, it is recommended that only one value be used, weighted by the shear resistance value [4].

To simplify the calculations under the bi-linear failure mechanism and determine the critical failure angle, $\theta_{1,}$ a series of design charts have been published. These charts provide for a given structure geometry L/H, both the critical angle and the Factor of

Safety for given shear strength characteristics of the soil (ϕ, c), facing inclination, reinforcement inclination, and loading condition (embankment slope, and surcharge). The charts are constructed for any assumed limit shear force per unit length of nail (T_m), per unit facing surface area defined as [1];

$$\mu = T_m / \gamma \, S_h \, S_v \tag{4.3}$$

where S_v and S_h are respectively the vertical and horizontal spacing of nails which must be assumed first, to obtain the FS directly from the charts.

Design charts using the bi-linear failure mechanisms for two common conditions encountered in practice are shown in figure 4.3.

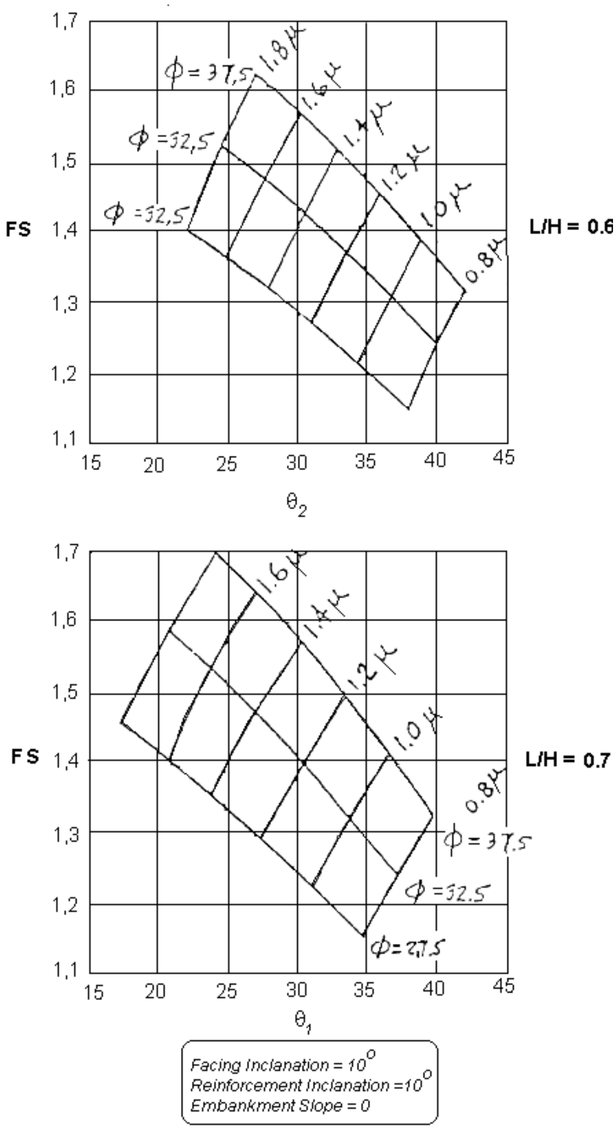


Figure 4.3 Diagrams for stability calculations, German Method [1]

4.2.1.1.2 The Davis Method (Shen, et.al., 1981)

This method, first developed at the University of California at Davis, is a limit equilibrium method, as well. It assumes that the potential failure surfaces are vertical axis parabolas, the vertices of which are located at the bottom of the facing (Figure 4.4) [4].

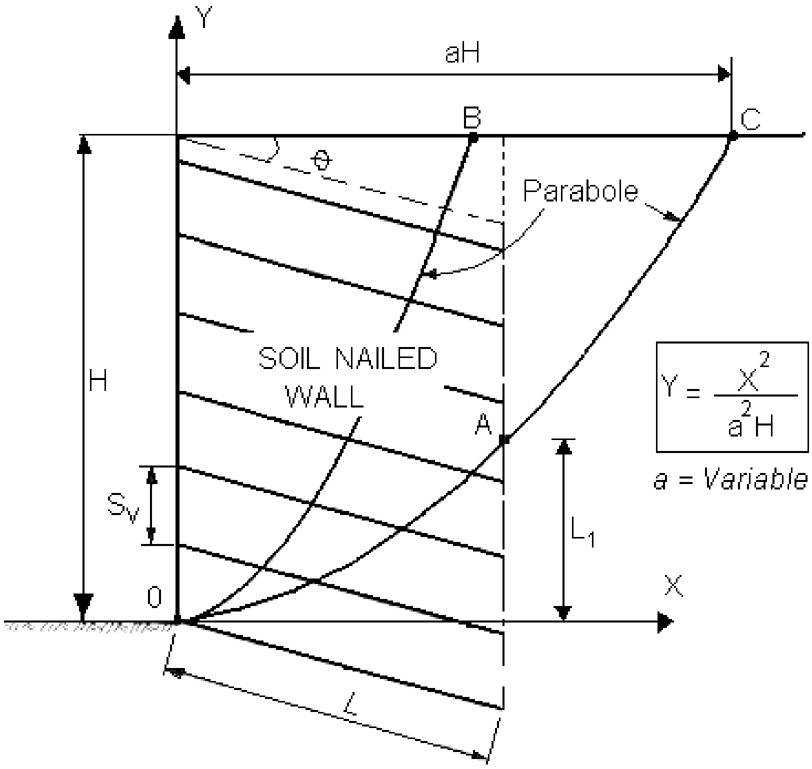


Figure 4.4 Possible failure surfaces, Davis Method [1]

The soil is assumed to be homogeneous and without water, and the geometry of the wall is simple (vertical facing, horizontal soil surface at the top, parallel nail rows, equidistant and of the same length). Only tensile forces are considered in the reinforcements. Breakage strength of the reinforcing members is calculated as the yield strength [1,4]:

$$T_b = A_s f_y \tag{4.4}$$

A_s = Cross-sectional area of the nail

f_y = Yield stress of the reinforcements

The pull-out resistance of each reinforcing member is calculated as a function of the frictional resistance along the adherence length of the nail in the passive zone. The frictional resistance is the shear stress mobilized between the nail and the soil as [1]:

$$T_p = \pi D_c L_a (\sigma_n \tan \phi_m + c_m) / S_h \quad (4.5)$$

T_p = Ultimate skin friction force

D_c = Diameter of grouted reinforcement

L_a = Adherence length of reinforcement in the passive zone

σ_n = Average normal stress on L_a

ϕ_m = Mobilized friction angle

c_m = Mobilized cohesion

S_h = Spacing of reinforcement

The resistance at failure T_R in every nail is taken as equal to the lowest resistance of either breakage by traction (T_b) or pull-out (T_p) [4].

$$\tau = \min (T_b , T_p) \quad (4.6)$$

The nail resistance for each row of reinforcements are added to produce an equivalent total tensile force which is considered in the global factor of safety analysis of driving and resisting forces.

Two failure surface conditions are considered. (1)The failure surface may lie entirely within the reinforced zone and (2) The failure surface may lie partially outside of the reinforced zone.

The overall stability of the excavation is evaluated through a limit equilibrium analysis of the driving weight force of the active zone and the resisting forces available along the assumed failure surface due to the soil reaction and nail forces. Simplifying assumptions with respect to force summation have been made to simplify computations and provide an overall factor of safety (FS).

The assumption is that the partial factors of safety with respect to the limit shear strength of the soil and pull-out resistance are all equal to the same global safety factor.

The lateral shear stress at the interface as calculated according to Mohr-Coulomb failure criterion [1]:

$$\tau_{mob} = \sigma_n \tan \phi / FS + c / FS \quad (4.7)$$

$\tau_{mob} = \min (T_b / FS , T_p / FS)$ (4.8)

Thus the minimum safety factor value corresponding to the most critical parabola can be calculated [4].

As with the German Method, two blocks separated by a vertical line passing through the extremity of the nails are examined when the failure surf ace exits beyond the reinforced volume. To calculate the forces between these two blocks, a coefficient κ is used, defined as the ratio of the horizontal and vertical stresses, and taken as equal to 0,4 in frictional soils and 0,5 for cohesive soils [4].

An iterative method, calculates the overall FS, which begins by assuming FS = L / H, then calculates the driving and resisting forces. The minimum Factor of Safety is then obtained [1].

Normally for design, a minimum overall factor of safety of 1,5 has been recommended [1].

4.2.1.1.3 The “Modified" Davis Method (Elias and Juran, 1988)

For preliminary design, Elias and Juran, using the Davis Modified Method, prepared the charts illustrated in figure 4.5. These charts represent solutions for four common geometries encountered in highway practice. For other geometries, interpolation may be used. The charts have been prepared for equal length of nails, a global factor of safety equal to 1,0 and a constant design cohesive strength equal to 7,5 kPa which represents the minimum cohesion necessary to construct stable initial cuts. In using these charts, the soil strength (ϕ) should be factored by 1,20, and the ultimate interface shear force (F_1) factored by 1,75 to 2,0 depending on the quality of the data in order to obtain structure dimensions consistent with the recommended factors of safety [5].

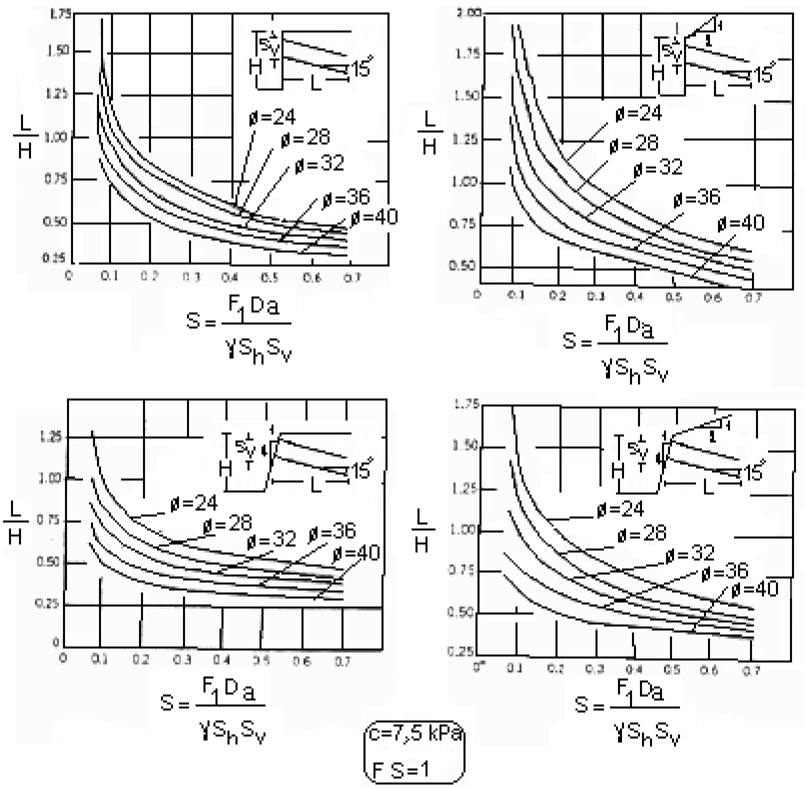


Figure 4.5   Modified Davis method design charts [5]

To use the design charts, the pull-out resistance is defined by the non-dimensional parameter [5]:

$$S = \frac{F_1 \, D_a}{\gamma \; S_h \, S_v} \tag{4.9}$$

where;

F_1 = ultimate interface shear force

D_a = assumed drill hole diameter

S_h = horizontal nail spacing

S_v = vertical nail spacing

γ = unit weight of the soil

For an assumed design S value, the charts are used with the factored frictional strength of the in-situ soil (ϕ) to obtain the corresponding L/H. The drill hole diameter and spacing can then be varied to arrive at the most economical design [5].

Note that the Davis analysis in its present form does not consider the effects of soil stratification and groundwater in section. Parametric studies using TALREN and the kinematical design method have indicated that static groundwater level in the lower half of the structure ($Z/H > 0,5$) have the effect of increasing L/H ratios on the order of 10 to 15 percent, but the effect is substantially more significant with higher water table.

4.2.1.2 Multi-Criteria Limit Equilibrium Analysis:

This approach, integrating the two fundamental mechanisms of soil-inclusion interaction (i.e., interface friction and passive normal soil reaction on the nail) was developed by Schlosser, (1983, the French Method). This solution allows for several definitions of the failure surface and for consideration of both tension and shearing resistance of the inclusion as well as the effect of their bending stiffness. The multi-criteria analysis is conducted to evaluate the global stability of the nailed-soil system with respect to four potential failure modes: shear failure of the soil along the critical sliding surface, pull-out failure of the nail, nail breakage by either excessive bending or combined effect of tension and shear forces, and creep or plastic flow of the soil between the nails [5].

This multi-criteria analysis procedure uses a conventional slices method that is modified to take into account the resisting nail forces in the equilibrium of each slice. Using the perturbation method the TALREN program, developed by Schlosser (1983) allows for any failure surface to be taken into account. This procedure permits an evaluation of the effect of soil stratification, groundwater flow, and seismic loading on the global structure stability. It can also be used for the design of mixed structures associating ground anchors and soil nailing [5].

4.2.1.2.1 The French Method (Schlosser, 1983)

The French Method considers the tensile resistance, shearing capacity and bending stiffness of the nails in evaluating their contribution to the overall stability of the in-situ reinforced soil mass. Overall stability is assessed along either circular or non-circular failure surfaces utilizing a specific method of vertical slices in computing the factor of safety along a potential sliding surface in a given section [1].

The design method recommended in France is the “Multi-criterium Method”, which is an extension of classical limit equilibrium methods (method of slices) to reinforced soils, allows the bending stiffness and the shear resistance of the nails to be taken into account when necessary [6]. For practical application of this multi-criteria analysis method, a computer program, TALREN 97(see Appendix), was developed in France and made available to the project team for evaluation and comparison. This multi-criteria analysis is performed by considering the following four potential failure modes.

4.2.1.2.1.1 Four Potential Failure Modes:

1. Lateral friction criterion:

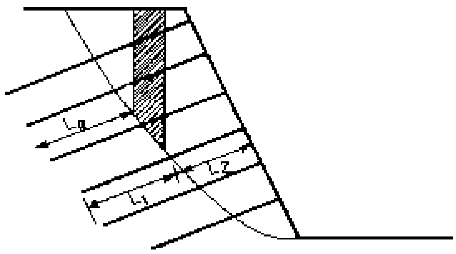
For a circular inclusion with diameter D, assuming that the limit interface lateral shear stress F_1 is constant all along the embedment length L_a, the tensile force can be evaluated as follows [4,18]:

$$T_n \leq F_1 \pi D L_a \tag{4.10}$$

πD is the perimeter of the nail where $D = D_c$ (borehole diameter) for grouted nails, and $D = D_a$ (equivalent diameter) for driven nails.

L_a is the nail grouted length beyond the failure surface, except where there is no facing. In this case $L_a = L^*$, the length L^* is the shorter of the two lengths between the failure surface and the facing (Figure 4.6).

This criterion correspond to a vertical line in the (T_n, T_c) diagram (Figure 4.7).



$L^* = L_a =$ the smaller of (L_1,L_2)

Figure 4.6 Determination of pull-out length[18]

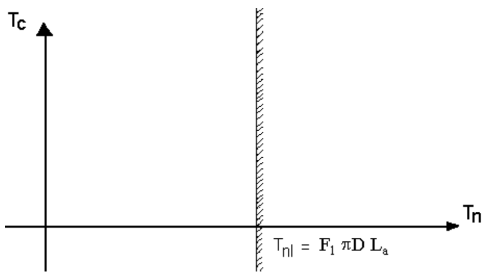


Figure 4.7 Stability domain corresponding to the soil-inclusion lateral friction[18]

2. Strength of the inclusion:

In soil nailed structures, the length of the reinforcement L is substantially greater than three times the transfer length, L_o. Theoretical solutions therefore suggest that the elastic nails can be considered as infinitely long and the relative displacement of the soil in the absence of the inclusion is therefore equal to the distance $2y_0$ between the two extremities of the inclusion as shown in figure 4.8 [1].

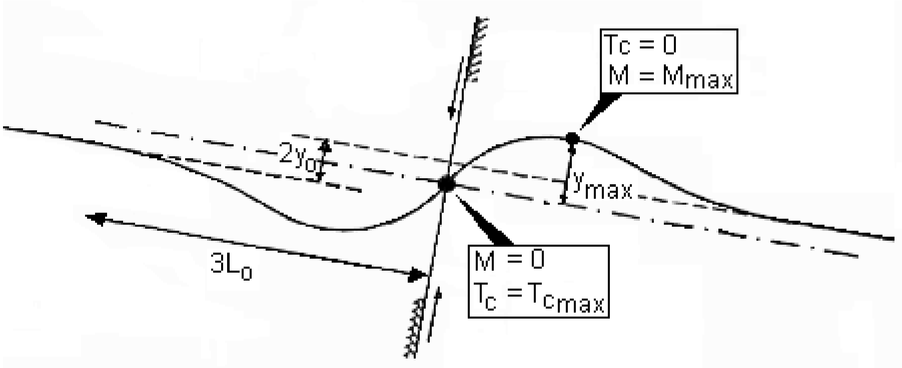


Figure 4.8 Bending of a rigid inclusion[1]

General Failure Criterion of the nail [4]:

$$\left(\frac{T_n}{R_n}\right)^2+\left(\frac{T_c}{R_c}\right)^2+\left|\frac{M}{M_o}\right|-1 \leq 0 \tag{4.11}$$

Nail plastification by shearing occurs at the point of maximum shear force 0 [4].

At the point 0, the bending moment is zero (M=0) and the failure criterion, based on the general failure criterion of the nail, can be written as [4];

$$\left(\frac{T_n}{R_n}\right)^2+\left(\frac{T_c}{R_c}\right)^2 \leq 1 \tag{4.12}$$

One usually takes;

$$R_c=\frac{R_n}{2} \tag{4.13}$$

The stability domain of the bar is delimited in the (T_n , T_c) plane by an ellipse with axes R_n and R_c , at the interior of which the vector $T(T_n, T_c)$ must be located (Figure 4.9).

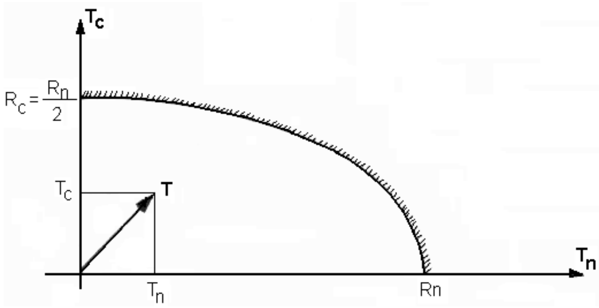


Figure 4.9 Stability domain of the steel, at the point of zero moment (point=0)[18]

3. Soil-Inclusion Normal Reaction:

During the relative displacement between the stable soil mass and the active zone, the inclusion deforms as indicated on Figure 4.10. The soil-inclusion normal pressure is maximum at the point of maximum displacement (point 0).

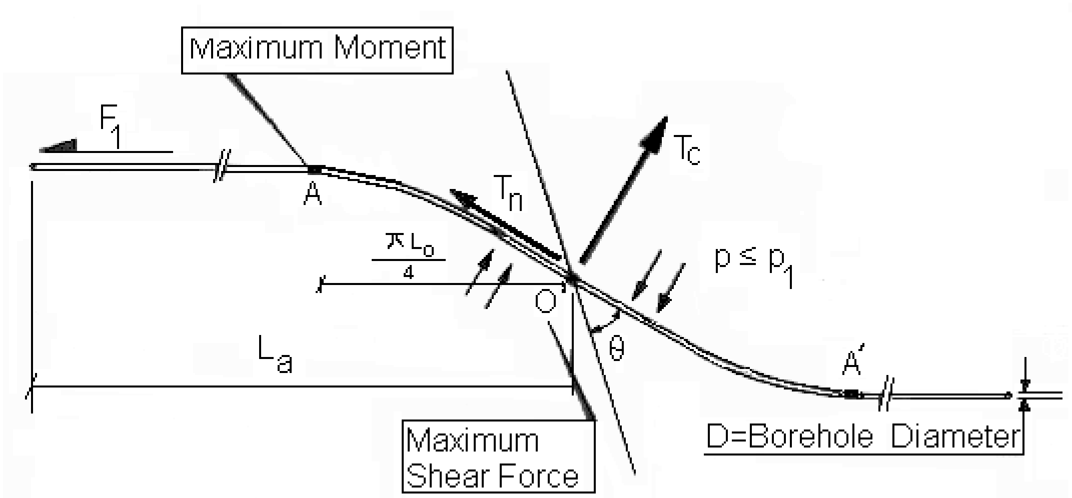


Figure 4.10 Procedure for taking into account the reinforcement [18]

- Criterion ($M_{max} \geq 0{,}16\ p_l\ D\ L_o^2$):

  During soil plastification at point 0 and in the absence of plastification within the bar, the limit shear force of the inclusion at this point can be obtained from [18]:

$$T_c = \frac{p_l\ D_c\ L_o}{2} \tag{4.14}$$

$$L_o = \left[\frac{4\ E\ I}{K_h\ D}\right]^{1/4} \tag{4.15}$$

This criterion is represented by a horizontal line in the (T_n, T_c) diagram (Figure 4.11).

An increase in the shear force above this value, corresponding an extension of the plastic zone, is not permitted [18].

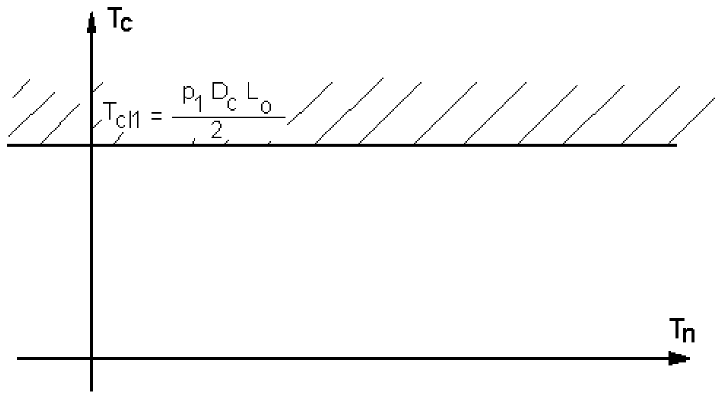


Figure 4.11 Stability domain resulting from the soil-inclusion normal force interaction at point 0, without plastification of the inclusion [18]

♦ Criterion ($M_{max} < 0{,}16\ p_l\ D\ L_o^2$):

Nail plastification by bending moment occurs at the point of maximum moment A and A' located on both sides of the potential failure surface at a distance equal to (Figure 4.10);

$$\frac{\pi}{4} L_o \tag{4.16}$$

When plastification of the inclusion occurs at point A, prior to soil plastification at point 0, the shear force at "0"is [18];

$$T_c = 1{,}62 \ \frac{M_{max}}{L_o} + 0{,}24\ D\ L_o\ p_1 \tag{4.17}$$

This criterion is of the parabolic type with a downward concavity in the (T_n, T_c) diagram (figure 4.12).

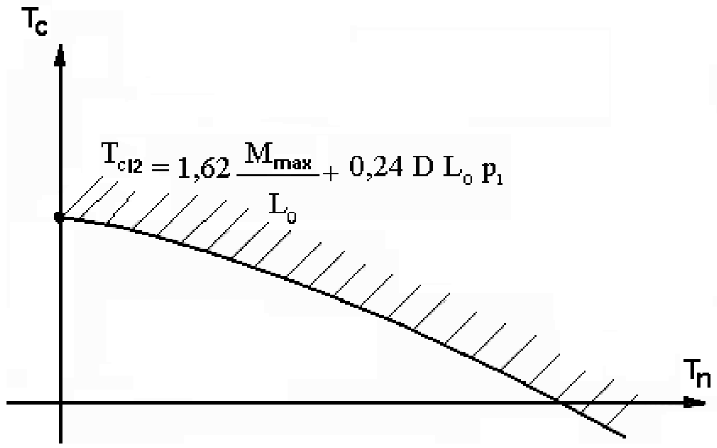


Figure 4.12 Stability domain of the bar at point A and of the soil taking into account the maximum plastification moment of the bar and the soil-inclusion normal interaction at point 0 [18].

4.2.1.2.1.2 Combinations of Failure Criteria:

The multi-criteria rule consists of representing the four criteria in the (T_n, T_c) plane. Their intersection is then considered to be the resultant criterion for the forces in the nails at point 0. The intersection of these criteria (figure 4.13) defines a convex domain of stability in which the representative point for the forces in the nail at failure at the point where it intersects with the potential failure surface can, at first, fall anywhere at the outer edge of the domain [4].

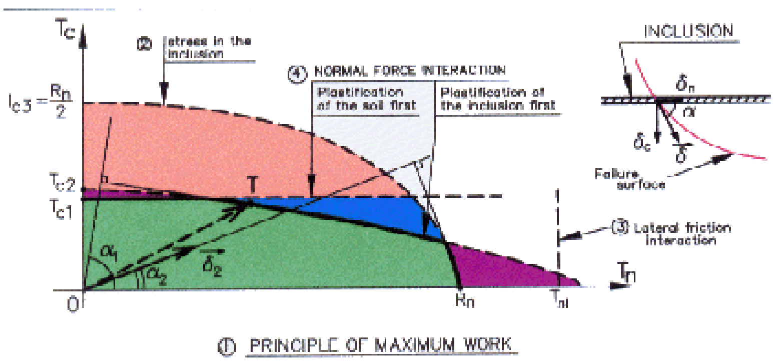


Figure 4.13 Combinations of failure criteria (Multi-Criteria Rule).
Determination of the forces in the nails [32]

For design purposes, bending moments and shear forces are often neglected. This can be acceptable if nails have a low moment of inertia (small cross-sections like in driven nails), are nearly horizontal (inclination to the horizontal lower than 20 degrees) and if there is not any surcharge on top of the wall [6].

The second concern when designing a structure, after making it stable, is to make it safe by incorporating a factor of safety. Traditionally, safety is considered globally with the help of a global factor of safety. For simple academic exercise, the failure surface can be modelled as a straight line defining a wedge as shown in figure 4.14. The equilibrium can be written under the following form: $FS = T_{max} / T$, with T_{max}, maximum resisting shear forces on the plane, T, projection of external forces on the plane [6].

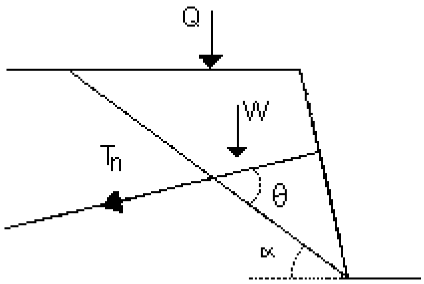


Figure 4.14 Simple analysis of a wedge failure using a global factor of safety[6]

The structure will be stable if FS is higher than 1,0 and it will be considered to be safe if FS is higher than 1,3 for a temporary structure and 1,5 for a permanent one [6]. The global factor of safety represents the margin of safety which is taken to account for uncertainties on the materials properties, on the loading conditions and errors inherent to the design method. In the partial safety factors approach, the usual global factor of safety is divided into a series of coefficients. The partial safety factors, Γ_m, applied on the strengths of materials, account for their variabilities and the uncertainties on their measurements. Typically for a soil, the friction angle ϕ will be better known than the cohesion c. Therefore, the partial safety factor on ϕ will have a value of 1,20 lower than the factor on c equal to 1,50. The weighing factors Γ_Q, applied on the external forces Q, account for uncertainties on the loading conditions. On gravity forces G, the partial factor is Γ_G equal to 0,95 to 1,05 to account for variations in the load. A method coefficient Γ_{S3} equal to 1,125 is introduced to account for errors inherent to the method. For the wedge failure mechanism, the structure will be safe if the following inequality is respected [6]:

$$\Gamma_{S3}\, T \leq T_{max} \tag{4.18}$$

with

$$T_n = \min \left(\frac{R_n}{\Gamma_{mRn}}, \pi D\, L_a \frac{F_1}{\Gamma_{mF1}} \right) \tag{4.19}$$

$$T = (W\,\Gamma_G + Q\,\Gamma_Q) \sin\alpha - T_n \cos\theta \tag{4.20}$$

$$T_{max} = [(W\,\Gamma_G + Q\,\Gamma_Q) \cos\alpha + T_n \sin\theta] \frac{\tan\phi}{\Gamma_{m\phi}} + \frac{c}{\Gamma_{mc}} (H/\sin\theta) \tag{4.21}$$

Ideally, partial safety coefficients and external forces weighing factors should be chosen based on observed dispersion laws of materials parameters and loading conditions while the other factors would be chosen to ensure a given maximum probability of failure (Gassler and Gudehus, 1983). However, because of the complexity of such statistic-probabilistical analyses and since the global safety factor method give acceptable design from the point of view of experience, partial factors are chosen to account for uncertainties as well as to give results in agreement with the experience (Table 4.2).

Table 4.2 Partial safety factors

External Forces	Weighing Factors
Weight G	Γ_G = 0,95 or 1,05
Surcharge Q	Γ_Q = 0,90 or 1,20
Soil Strength	Partial Safety Factor
Friction tan	$\Gamma_{m\phi}$ = 1,20
Cohesion	Γ_{mc} = 1,50
Soil Nail Interface	Partial Safety Factor
Frictional Resistance	Γ_{mF1} = 1,40
Steel Nail	Partial Safety Factor
Tensile Strength	Γ_{mRn} = 1,15
Method Coefficient	Γ_{S3} = 1,125

The most important factor in the design of soil nailed structures is F_1: soil-nail interface frictional resistance. For design, F_1 should be determined by pull-out tests (Part 3).

In particular, the French National Research Project CLOUTERRE (1991) provided a data base to suggest preliminary design charts that yield correlations between the unit skin friction (F_1) and the pressure limit p_1 obtained with the pressuremeter in different types of soils for both driven nails and gravity grouted nails [4].

The unit skin frictions (F_1) values are taken from the following table and reported in figures 4.15 to 4.19.

Table 4.3 Correspondence between the charts, the soils, and the construction techniques [4]

SOILS	CORRESPONDING CHARTS	CONSTRUCTION TECHNIQUE		
		Gravity Grouting	Low Pressure Grouting	Driving
Sand	Figure 4.15	S1	G2	S3
Gravel	Figure 4.16	G1		G3
Clay/Silt	Figure 4.17	A1		
Marl-Chalk	Figure 4.18	M1		
Weathered Rock	Figure 4.19	R1		

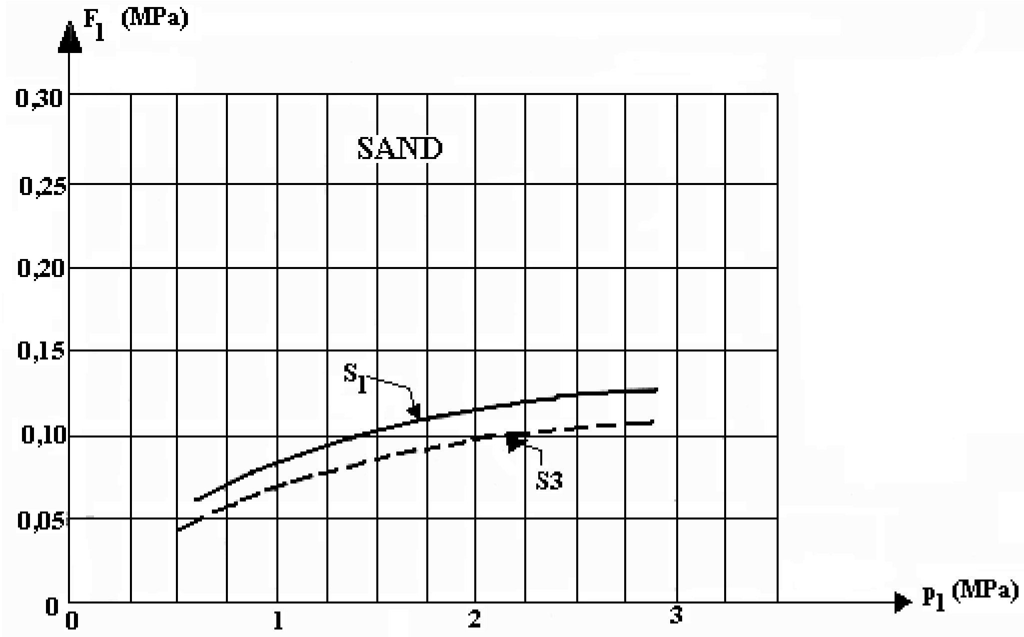


Figure 4.15 Chart to estimate the unit skin friction F_1 for sand [4]

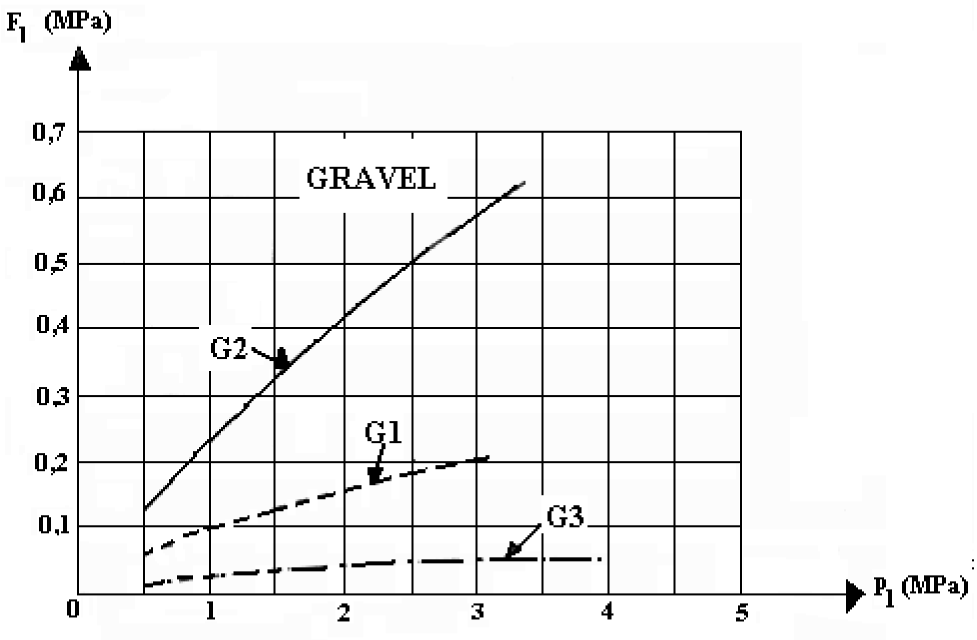


Figure 4.16 Chart to estimate the unit skin friction F_1 for gravel [4]

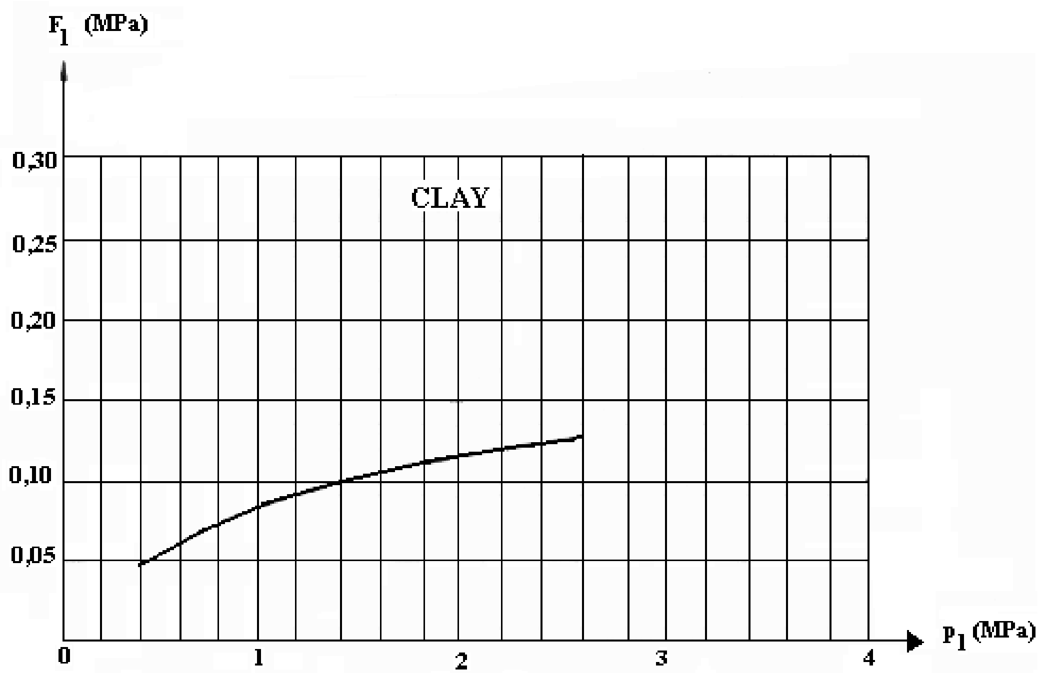


Figure 4.17 Chart to estimate the unit skin friction F_1 for clay [4]

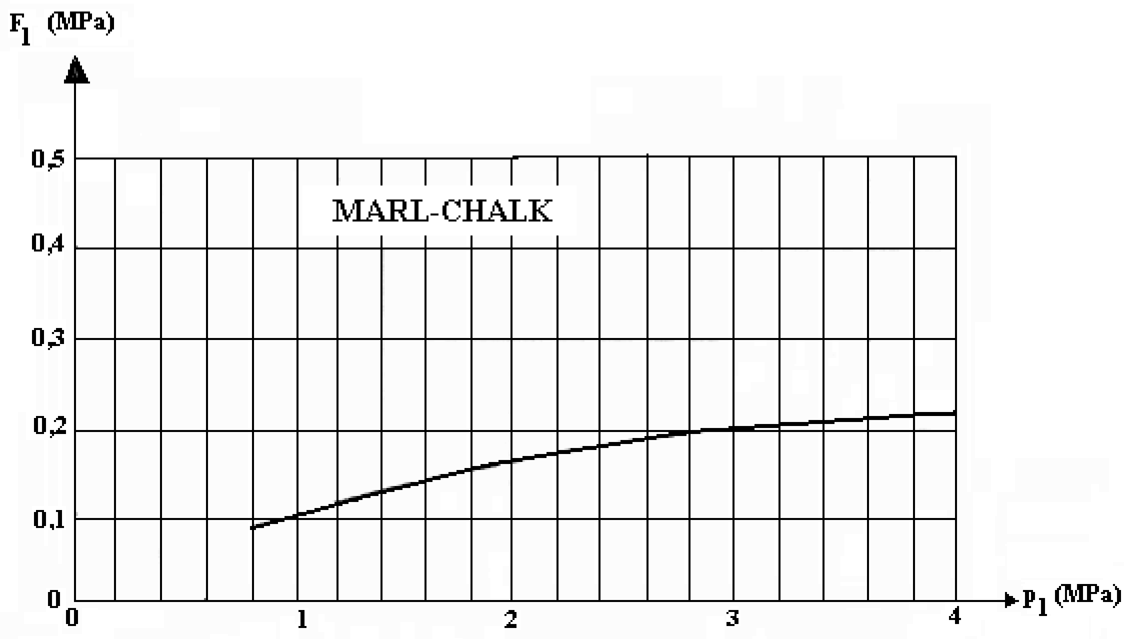


Figure 4.18 Chart to estimate the unit skin friction F_1 for marl-chalk [4]

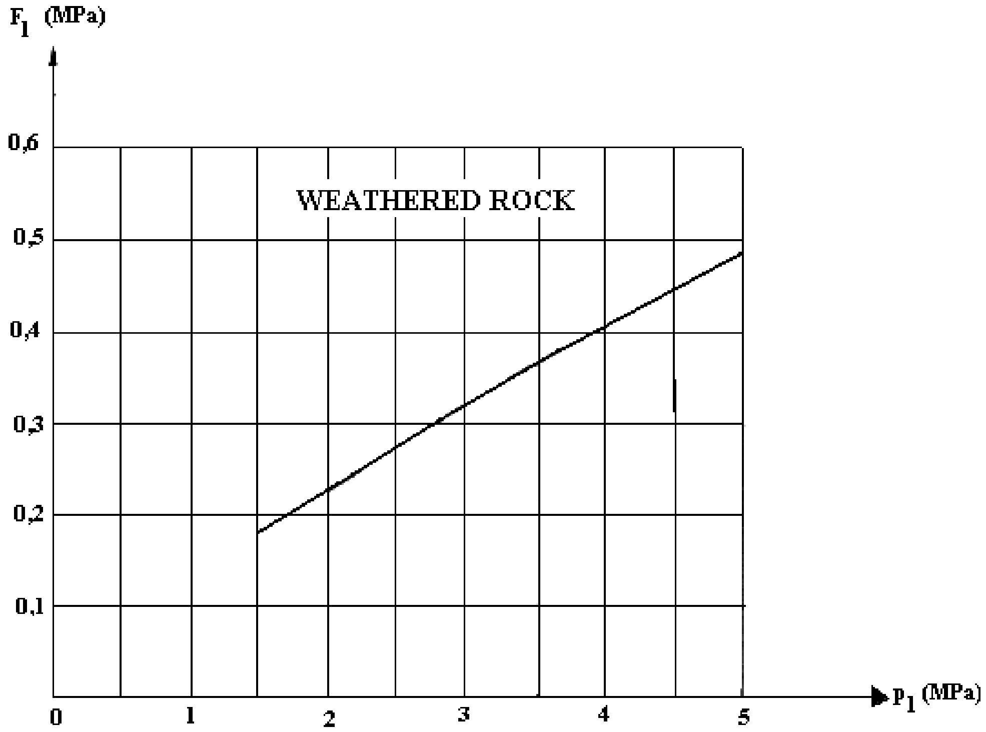


Figure 4.19 Chart to estimate the unit skin friction F_1 for weathered rock [4]

4.2.2 Working Stress Design Methods

Several approaches have been developed to estimate nail forces in nailed soil-retaining structures. They can be broadly classified into three main categories [5]:

a. Empirical design-Earth pressure diagrams:

b. Finite-element analysis

c. Kinematical limit analysis

4.2.2.1 Empirical Design - Earth Pressure Diagrams

Selection of an appropriate earth pressure diagram for the determination of the nail forces should be consistent with the nature of the retained soils and developed ground movements. Measurements of facing displacements in soil nailed structures suggest that in non-plastic soil, these displacements are comparable to those measured in braced excavations. Therefore, design diagrams proposed by Terzaghi and Peck and Tschebotarioff for the design of braced excavations, provide a rational estimate of working tensile forces generated in the nails. These diagrams are schematically illustrated in figure 4.20. The design diagram for sands has been slightly modified in order to calculate nail forces. The maximum tension force mobilized in the nail is expressed as a normalized non-dimensional parameter [1]:

$$TN = \frac{T_{max}}{\gamma H S_h S_v} \qquad (4.22)$$

- *For sands* (c/γH < 0,05):

$$TN = 0{,}65\ K_a \qquad (4.23)$$

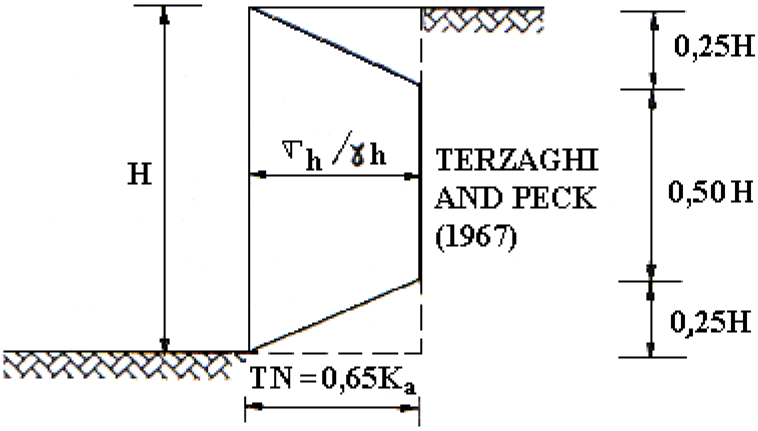
$$K_a = \tan^2 (45-\phi/2) \qquad (4.24)$$

- *For cohesive soils* with both cohesion (c) and friction angle :

$$TN = K_a \left[1 - \frac{4c}{\gamma H} \left(\frac{1}{K_a} \right)^{1/2} \right] < 0{,}65\ K_a \qquad (4.25)$$

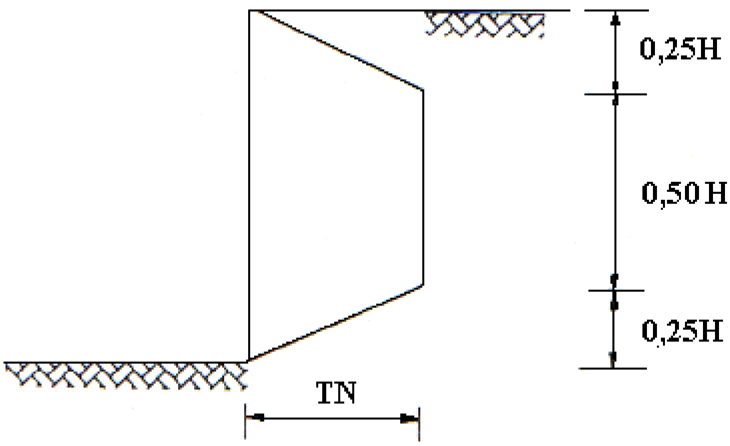
The validity of this empirical simplification has been demonstrated for certain limited cases. The measured nail forces, for grouted nails, are found to agree fairly well with the assumed earth pressure design diagrams. These results suggest observed behavior of nailed cut slopes is similar to that of braced excavations.

However, the use of the empirical earth pressure diagrams for design of soil nailed retaining structures presents some limitations. In particular, these diagrams correspond to conventional cases of bracing supports with the simple geometry of a vertical wall, horizontal ground surface and lateral braces. Therefore, they can not be used to asses the effect of design parameters such as inclination of the facing, inclination and rigidity of the nails, surcharge, groundwater, etc. on the working forces in the nails and the structure displacements. They do not provide any data with regard to the shear forces and bending moments that can develop in the nails. In cohesive or mixed soils, the empirical earth pressure diagram is highly sensitive to small variations in soil properties.



SAND : $c / \gamma H \leq 0{,}05$

$$K_a = \tan^2 (45 - \varphi/2)$$



CLAYEY SAND : $TN = K_a \left[1 - \frac{4c}{\gamma H} \left(\frac{1}{K_a} \right)^{1/2} \right] \leq 0{,}65\ K_a$

CLAY : $TN = 0{,}2\ \gamma H \longrightarrow 0{,}4\ \gamma H$

<u>NOTES</u> :

- Vertical Cut Slope
- Horizontal Upper Surface
- σ_h: Lateral Earth Pressure
- γh : Overburden Pressure

Figure 4.20 Earth pressure diagrams for empirical design [1]

4.2.2.2 Finite-Element Analysis

During the past decade, the finite-element method has been used by several investigators to analyze the behavior of the soil-nailed retaining structures under both static and seismic loading conditions. These analyses involve different constitutive equations for the soil and interface elements to simulate soil-wall and soil inclusion interaction. In particular, attempts have been made by these investigators to compare finite-element predictions with observed behavior of instrumented structures. However, the use of finite-element methods in design is currently limited by the relatively high cost and raises significant difficulties with regard to the following [5];

1. The actual construction stages and installation process of the inclusions are difficult, if not practically impossible, to stimulate.
2. The complex soil-inclusion and soil-wall interaction is difficult to model. Several interface models have been developed, but their implementation in design requires relevant interface properties that are difficult to determine properly.
3. Various elastoplastic soil models can presently be used to predict soil behavior during excavation. However, determination of soil model parameters generally requires specific and rather elaborated testing procedures, which limits the practical use of these models.

The finite element method has therefore been used primarily as a research tool to evaluate the effect of the main design parameters on the engineering behavior of the structure, ground movement, and the working forces in the inclusions.

4.2.2.3 Kinematical Limit Analysis:

This limit analysis approach was developed initially (Juran et al., 1977) for the design of reinforced earth structures and adapted (Juran and Beech 1984, Juran 1987) for the design of nailed soil-retaining structures. It allows for the estimation of nail forces developed at serviceability limit state and the evaluation of the effect of the main design parameters on the tension and shear forces generated in the nails during construction [5].

This method is theoretically and numerically complex, has not yet been presented in a form that can be easily understood or used by practicing engineers, and has been challenged by others as containing questionable theoretical assumptions.

The design assumptions shown in figure 4.21 imply that;

1. The potential failure surfaces are taken to be logarithmic spirals intersecting the bottom of the wall.
2. The nailed mass is divided into slices parallel to the nails.
3. The horizontal component E_h of the force between ant two slices remains constant.
4. At failure, the locus of maximum tension and shear forces coincide with the failure surface developed in the soil.
5. The shearing resistance of the soil, as defined by Coulomb's criterion, is entirely mobilized all along the failure surface.
6. The shearing resistance of stiff inclusions is mobilized in the direction of the sliding surface in the soil and is defined by Tresca's criterion.
7. The effect of a slope (or horizontal surcharge) at the upper surface of the nailed soil mass on the forces in the inclusions is linearly decreasing with depth along the failure surface.

This method is interesting in that it can, by considering the local equilibrium in each slice, be used to calculate the tensile and the shear forces developed in each nail row of their point of intersection with the failure surface. Thus, the soil nailed wall can be designed to avoid any risk of progressive failure through the failure breakage beginning with rupture in one nail row.

At the failure surface, the bending moment in the nail is zero whereas the tension and shear forces are maximum. A normalized non-dimensional bending stiffness parameter, defined as [1]:

$$N = \frac{K_h D \ L_0^2}{\gamma H \ S_h S_v} \tag{4.26}$$

The bending stiffness parameter N for most practical structures vary from 0,1 to 1,5 [1].

where :

$$L_0 = \left[\frac{4 \ E \ I}{K_h D} \right]^{1/4} \tag{4.27}$$

H : Height of the wall

D : Diameter of the nail

S_h : Horizontal spacing of the nail

S_v : Vertical spacing of the nail

K_h : Modulus of lateral soil reaction (K_h may be estimated using charts developed for anchored walls, as a function of soil strength parameters shown in figure 4.22)

γ : Unit weight of the soil

L_0 : The transfer length

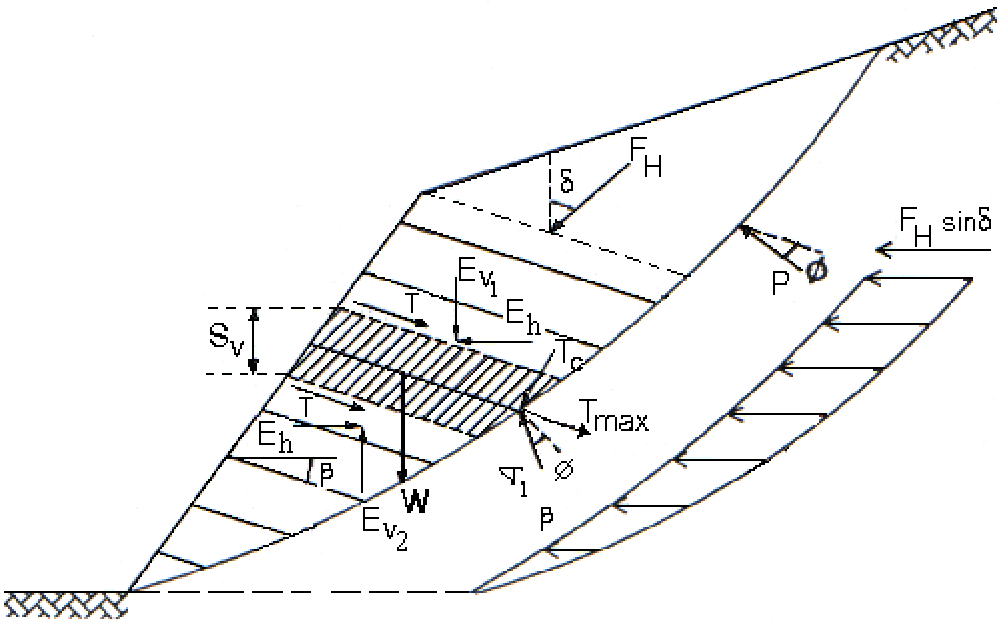
The unique failure surface which verifies all of the equilibrium conditions of the active zone can be defined. In order to establish the geometry of this failure surface, it is necessary to determine its inclination with respect to the vertical and the intersection with the upper ground surface. Observations on full-scale structures and laboratory model walls have been shown that for relatively flexible nails, (N < 1) the failure surface is practically vertical at the upper part of the structure [1].

The normal soil stress along this failure surface is calculated by Kotter's equation. The maximum tension force (T_{max}) in each nail is calculated from the horizontal force equilibrium of the slice containing the nail. Analysis of the state of stress in the nail yields the ratio of the mobilized shear (T_c) to tension (T_{max}) forces as a function of the nail inclination with respect to the failure surface.

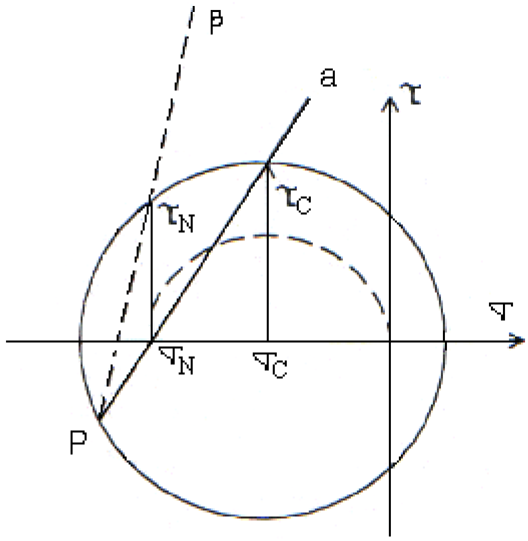
The numerical stability analysis proceeds by searching a unique failure surface which verifies all the equilibrium conditions. This approach provides an estimate of the locus and values of the maximum tension and shear forces mobilized in the nails at each level denoted non-dimensionally by Z/H. The values of the maximum tension force (T_{max}) and of the maximum shear force (T_c) actually mobilized are represented as normalized non-dimensional parameters:

$$TN = \frac{T_{max}}{\gamma H \, S_h S_v} \tag{4.28}$$

$$T_{max} = T_n \tag{4.29}$$



Mechanics of Failure and Design Assumptions



$$T_{max} = \sigma_N A_s$$

$$T_c = \tau_N A$$

A_s = Section Area

State of Stress in the Inclusion

Figure 4.21 Kinematical limit analysis approach [1]

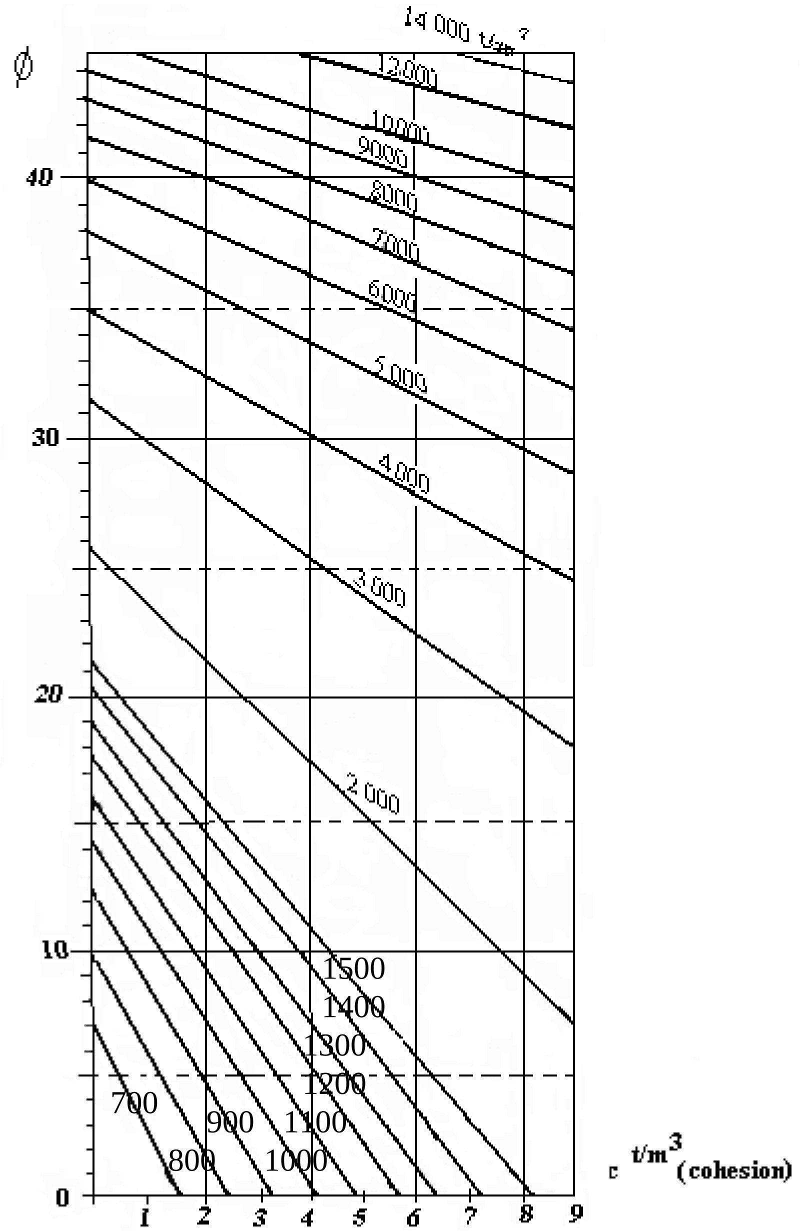


Figure 4.22 Horizontal subgrade reaction as a function of the soil shear strength parameters [1]

Design by the kinematical approach is based on the evaluation of local stability of each reinforcement with respect to these main failure criterias:

1. *Failure by pullout of the reinforcement:*

The relationship to be verified for this mode of failure is governed by :

$$TS = \frac{T_c}{\gamma H S_h S_v} \tag{4.30}$$

where:

$$\frac{T_{max}}{\pi D_c L_a} < \frac{F_1}{FS_P} \tag{4.31}$$

T_{max} : Maximum tensile force in the nail

L_a : Adherence length

FS_P : Safety Factor with respect to pullout.

F_1 : Limit interface lateral shear stress

D_c : Diameter of the grouted drill hole

This design criteria implies that for a soil nailed structure, the geometry defined by the L/H ratio should be verified at each reinforcement level such that [1]:

$$\frac{L}{H} \geq \frac{S}{H} + \left[FS_P \frac{TN}{\pi \mu} \right] \tag{4.32}$$

where;

S : Nail length in the active zone

L : Total nail length

2. *Failure by breakage of the reinforcement:*

For <u>flexible nails</u> (N=0) which withstand only tension forces [1]:

$$\frac{f_y A_s}{\gamma H S_h S_v} \leq TN \tag{4.33}$$

f_y : Yield stress of the reinforcement

A_s : Cross-sectional area of the nail

γ : Effective unit weight of the soil

For rigid nails (N > 0) which can withstand both tension and shear forces, considering Tresca's failure criterion [1]:

$$\frac{f_y \; A_s}{\gamma \; H \; S_h \, S_v} \geq K_{eq} \tag{4.34}$$

$$K_{eq} = \left[TN^2 + 4 \; TS^2 \right]^{1/2} \tag{4.35}$$

3. Failure by excessive bending:

For reinforcement having a defined bending stiffness, failure by breakage can also occur theoretically at the point of maximum moment when M_{max} exceeds the plastic moment of the reinforcing material M_p. Therefore, to prevent failure by excessive bending the following should be verified [1]:

$$M_p > FS_m \, M_{max} \tag{4.36}$$

where;

FS_m : A factor of safety with respect to plastic bending. Where allowable stress is used to design tension in the nail, use FS_m= 1 otherwise use FS_m= 1,8.

M_p : Plastic bending moment of the nail.

M_{max} : The bending moment. It is derived from the "p-y" analysis:

$$M_{max} = 0{,}32 \; T_c \; L_0 \tag{4.37}$$

$$\frac{M_p \; / \; L_0}{\gamma \; H \; S_h \, S_v} \geq 0{,}32 \;\; FS_m TS \tag{4.38}$$

This criteria should not be considered as governing due to the uncertainty in plastic moment computation if the nail is not well centralized. However, it does give an indication of minimum grout cover needed structurally for a given nail section [1].

The shear force in the inclusion should not exceed:

$$T_c = p_1 L_0 D / 2 \qquad (4.39)$$

The kinematical limit analysis approach provides prediction of magnitude and location of maximum tension and shear forces that are developed in the nails at working stress. It can therefore be evaluated by comparing predicted working stress nail forces with those measured in instrumented structures and model tests.

Local stability can be therefore evaluated using the following iterative procedure [1]:

1. Select the nail type, bending stiffness (EI), ultimate tension stress (F_y), diameter (D), and spacings (S_v, S_h).
2. Determine the ultimate friction limit (F_1).
3. Select an appropriate factor of safety for each soil strength parameter and allowable tension stress.
4. Determine the non-dimensional parameters.
5. Compute the required steel section for each height considered.
6. Verify that the selected reinforcement satisfies the breakage/excessive bending failure criteria.
7. Determine minimum required L/H ratio at each nail level based on pull-out stability. The ultimate friction limit F_1 should be factored by the chosen factor of safety against pull-out.

Detailed analyses at every level and computations considering all appropriate parameters at each depth are extensive and require a computer code for design optimization.

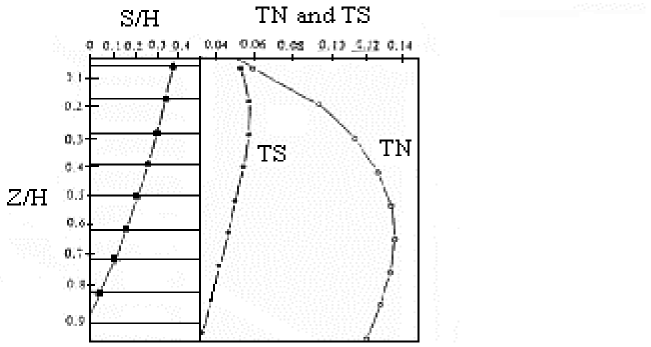
For design purpose, using the kinematical analysis program, the local stability analysis of the soil nailed structure at the level of each nail should satisfy the internal failure criteria summarized in Table 4.4.

Table 4.4 Internal failure criteria for nailed soil retaining structures [5]

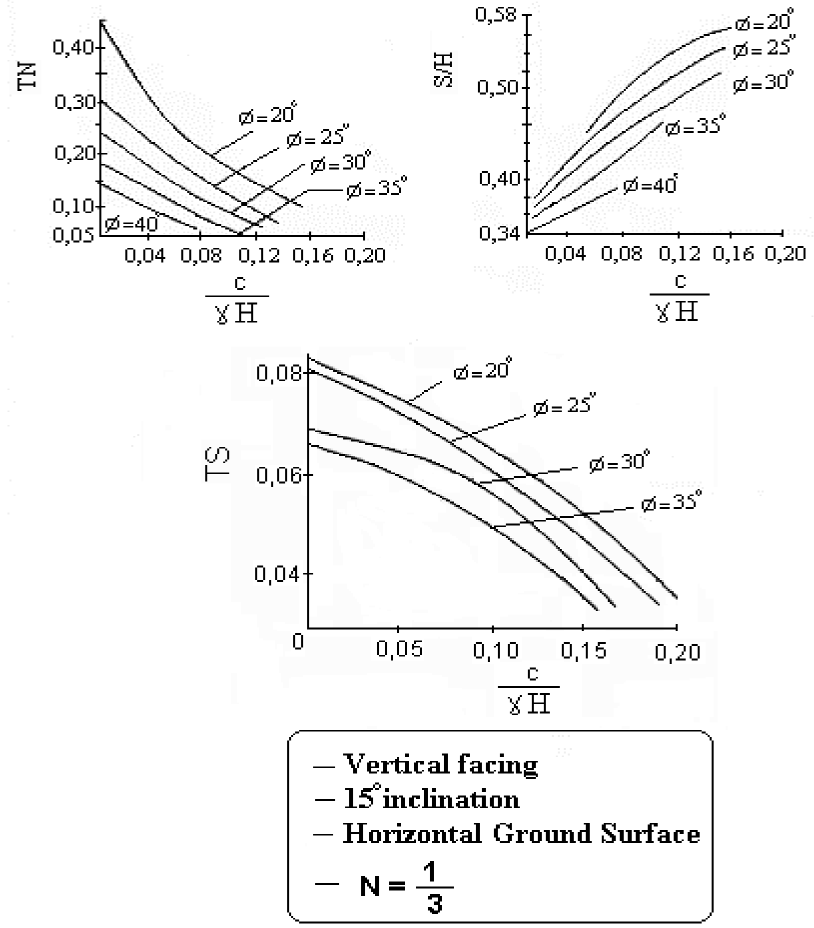
Pull-out Failure	***Breakage Failure***	***Failure by Excessive Bending***
$\frac{F_1}{FS_p} > \frac{T_{max}}{\pi D_c L_a}$ This design criterion implies that for a soil-nailed cut slope, the structure geometry defined by the L/H ratio should satisfy $\left[\frac{L}{H}\right] \geq \left[\frac{S}{H}\right] + FS_p \left[\frac{TN}{\pi \mu}\right]$ where; $\mu = \frac{F_1 D_g}{\gamma S_h S_v}$ $TN = \frac{T_{max}}{\gamma H S_h S_v}$	For Flexible Nails: $\frac{f_y A_s}{\gamma H S_h S_v} \leq TN$ For Rigid Nails: $\frac{f_y A_s}{\gamma H S_h S_v} \geq K_{eq}$ where; $K_{eq} = \left[TN^2 + 4.TS^2\right]^{1/2}$ $TS = \frac{T_c}{\gamma H S_h S_v}$ and T_c is the maximum shear force in the inclusion	$M_p > FS_m \cdot M_{max}$ The bending moment M_{max} is derived from the p-y analysis: $M_{max} = 0,32 .T_c \cdot L_o$ where; $L_o = [(4EI)/(K_h D)]^{1/4}$ is the transfer length, which characterizes the relative stiffness of the inclusion to the soil.

For preliminary design in homogeneous soils design charts have been prepared (Juran and Elias 1991). Design charts have been prepared for design of structures with uniform nail length, by considering the maximum S/H, TN and TS for each structure. These design charts established for the common geometry of vertical facing and horizontal ground surface considering perfectly flexible nails with 15^0 inclination. Figure 4.23 shows the type of graph proposed to calculate T_n and T_c knowing the value of the non-dimensional parameter N.

Analyses using the kinematical method suggest that with inclined reinforcements the bending stiffness can significantly effect nail forces. Nail forces in structures with horizontal backfills decrease with increasing stiffness (N>0), while in structures with sloping surcharges, the nail forces increase with increasing stiffness (N>0). Figure 4.24 illustrates the effect of nail inclination and the bending stiffness on TN, TS and S/H.



(a)



(b)

Figure 4.23 (a) Typical example of design output provided by the kinematical limit analysis approach (b) Charts used to calculate T_n , T_c , and S/H [5]

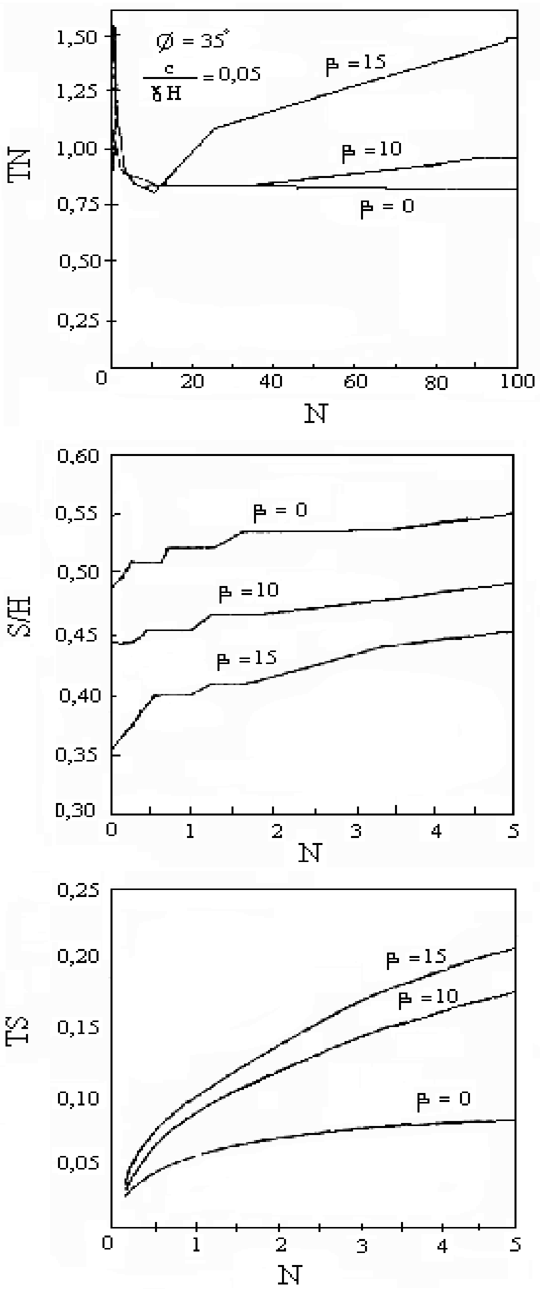


Figure 4.24 The effect of nail inclination and the bending stiffness on TN, TS and S/H

4.2.3 SEISMIC DESIGN

The following guidance is recommended in defining the appropriate design seismic coefficient [3]:

1. Select the appropriate design earthquake peak ground acceleration A_{pk}.
2. For slip surfaces that are primarily “internal” in nature, define a design seismic coefficient $A = (1,45 - A_{pk})\, A_{pk}$ [3]. The definition of “internal” is given on figure 4.25.
3. For slip surfaces that are primarily “external” in nature, the design seismic coefficient “A” will vary depending on the permanent displacements that are retaining wall can tolerate during the design event. For example, if the wall can tolerate permanent displacements of up to $250A_{pk}$ mm, then a design seismic coefficient equal to $0,5A_{pk}$ can be assumed.

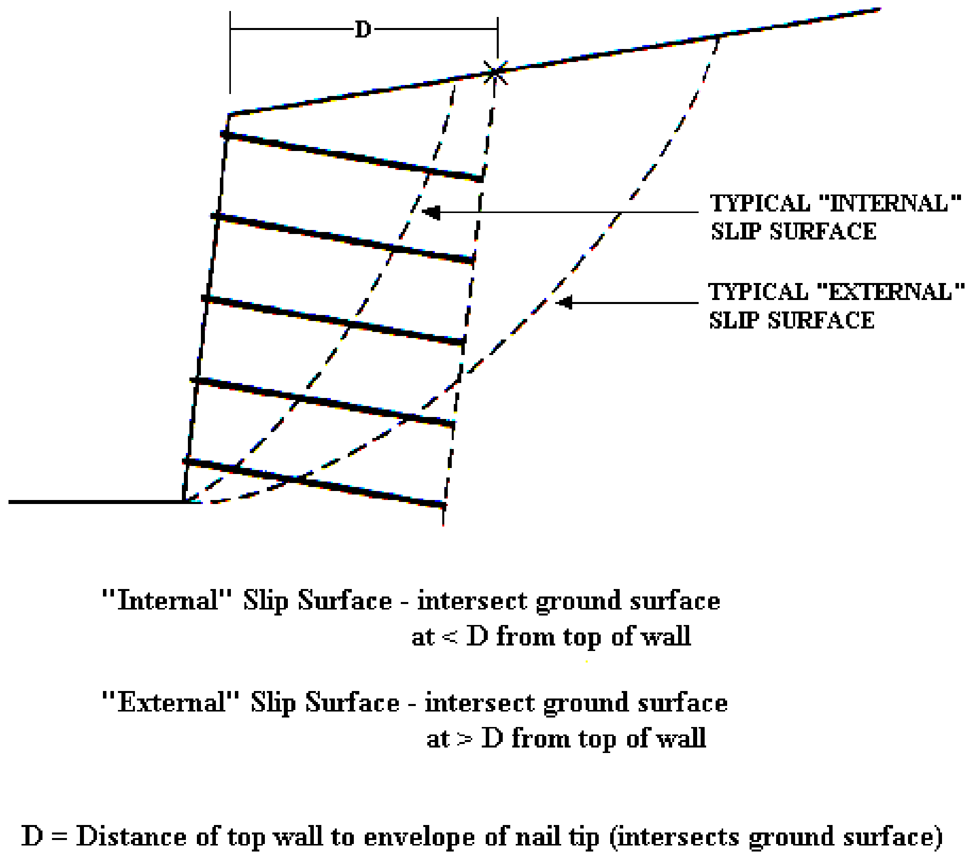


Figure 4.25 Definition of “Internal” and “External” Slip Surfaces for Seismic Loading Conditions [3]

5.SOIL NAIL WALLS OF ANATOLIAN MOTORWAY

5.1 EXCAVATIONS BETWEEN KM 14+800 – KM 15+187 (LEFT CARRIAGEWAY)

Gumusova – Gerede Section of Anatolian Motorway between 11+750 and 15+360 (Bolu Tunnel Portal) had been designed as reinforced concrete viaducts with L=40,0m beam spans. After November 1999 Duzce Earthquake, the owner has decided to built 3 viaducts and 2 fills above culverts in between these viaducts for this section. In this new solution for Asarsuyu crossing, excavations at the northern slopes at the toe of the historical Kom landslide have been avoided. Motorway platforms rest on controlled fills to be placed on a box culvert.

The left carriageway of the motorway is between KM 14+752 – 15+348. Between KM 14+800 and KM 15+187, the left carriageway of the motorway is in excavations. Five excavation zones can be identified depending on soil types and excavation heights. A longitudinal cross-section showing zones of excavation is illustrated in figure 5.1.

5.1.1 Excavations in North Slopes Between KM 15+060 – KM 15+187

Excavations along this section of the motorway shall be done in completely weathered rocks with slope debris overburden. Excavation heights shall be in the range of H= 8,52 m to 26,6 m if a 4V/1H slope is employed (Figure 5.2).

Alternative supporting systems have been considered for this section to assure slope stability. After studying all alternative solutions, it has been concluded that soil nailing is the optimum solution for these cuts.

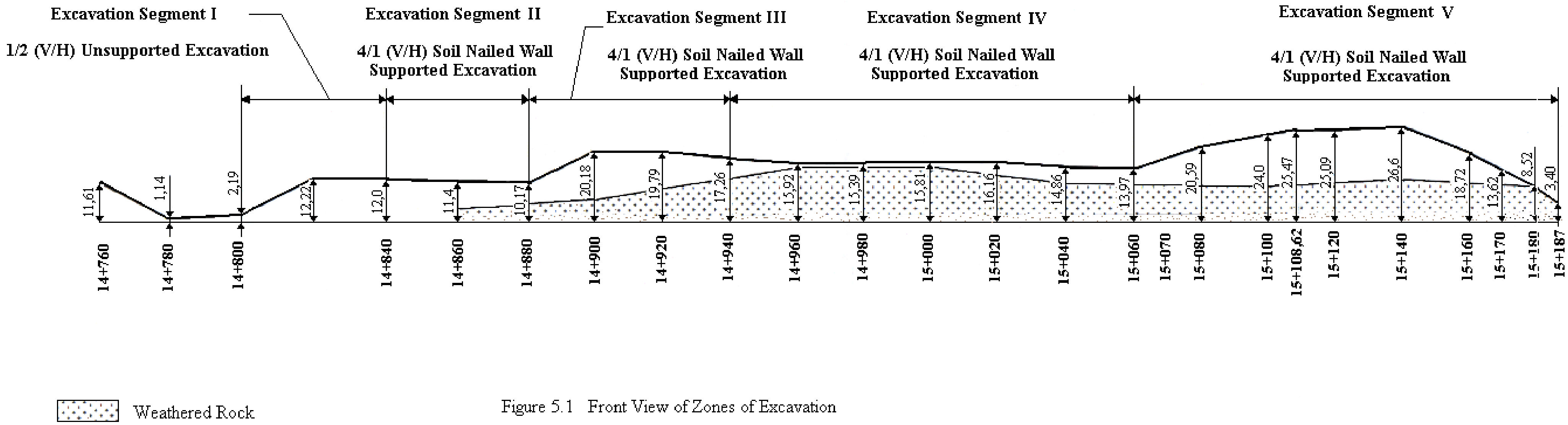


Figure 5.1 Front view of zones of excavation

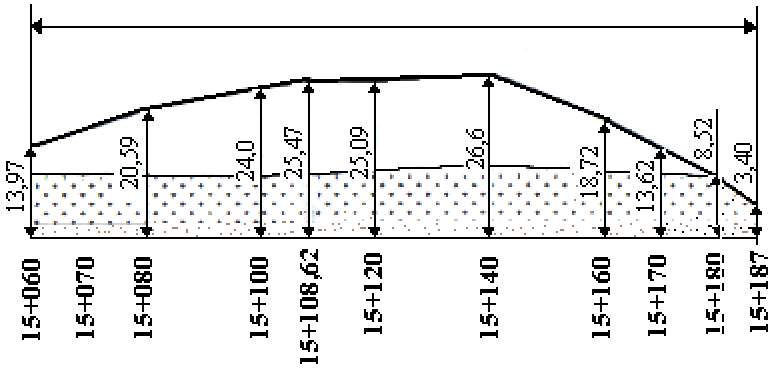


Figure 5.2 Excavations in North Slopes between KM 15+060 – KM 15+187 [21]

5.1.1.1 Geotechnical Investigations and Engineering Properties of Soils

1 horizontal and 1 vertical boreholes NND4 and NNB4 have been drilled to collect soil data for designing cuts in north slopes. Locations of borings are shown in figure 5.3. These borehole logs are shown in figure 5.4.

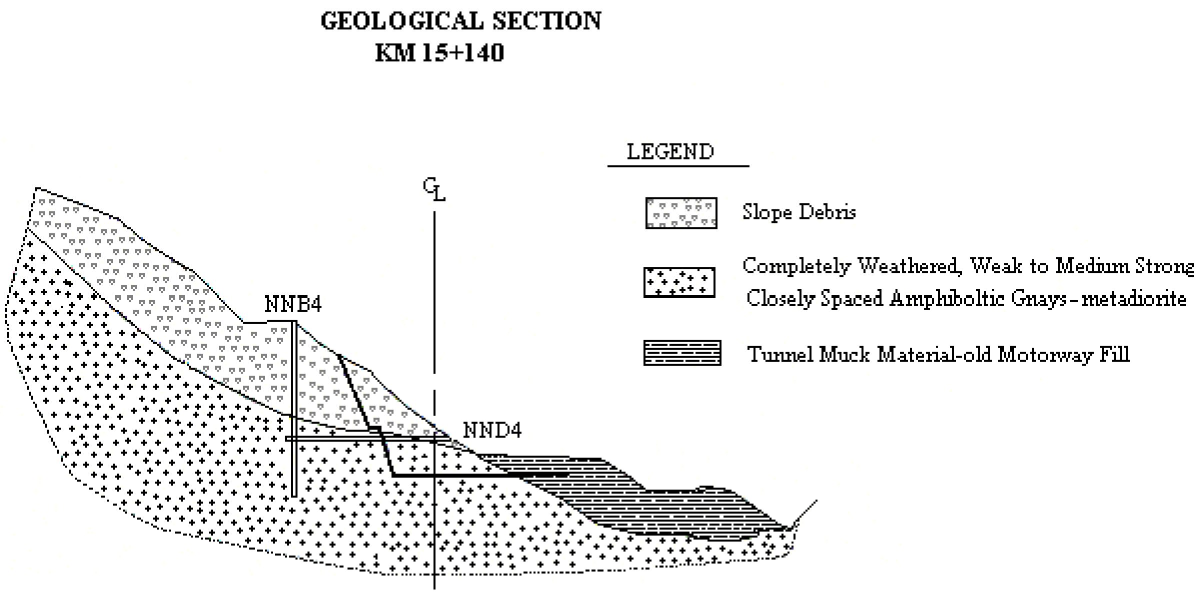


Figure 5.3 Locations of borings and soil profile at KM 15+140 [21]

BOREHOLE NO : NNB4

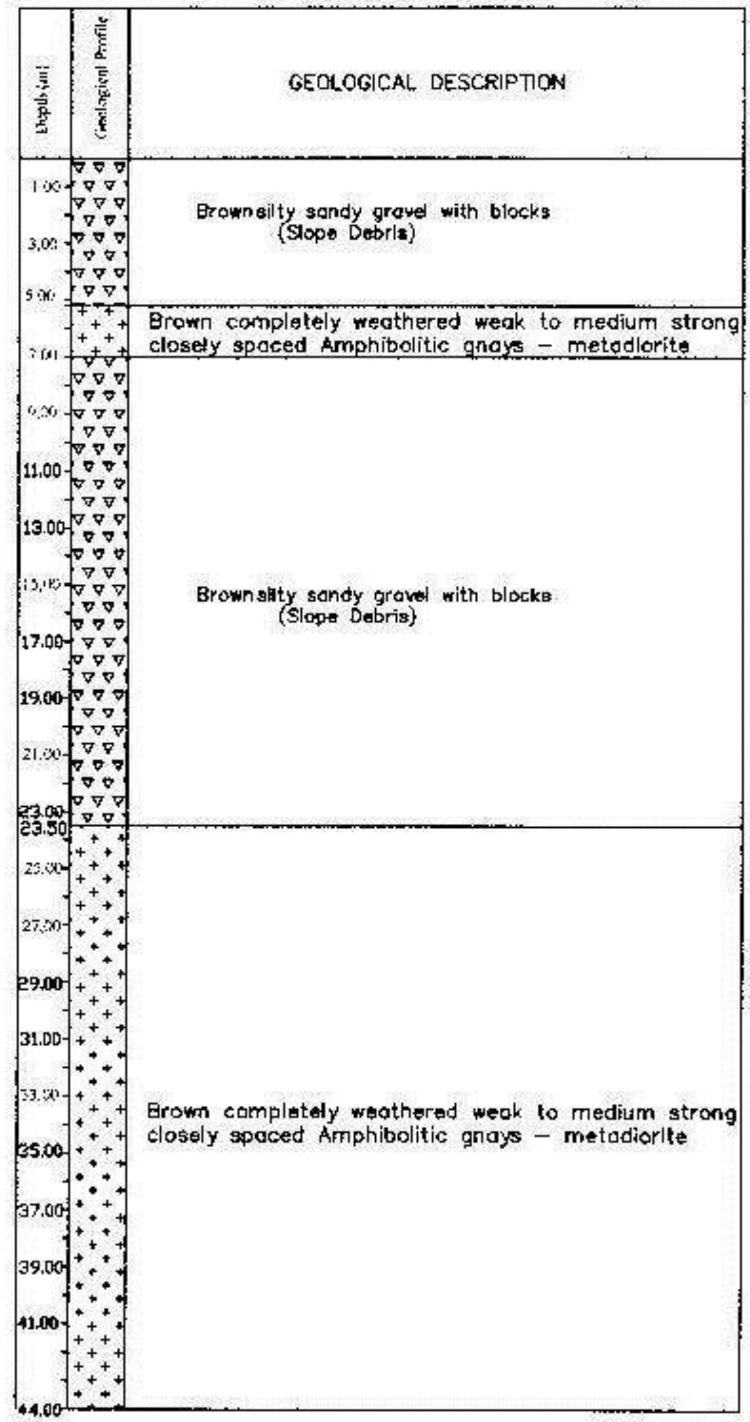


Figure 5.4 Borehole logs (NNB4 and NND4) [21]

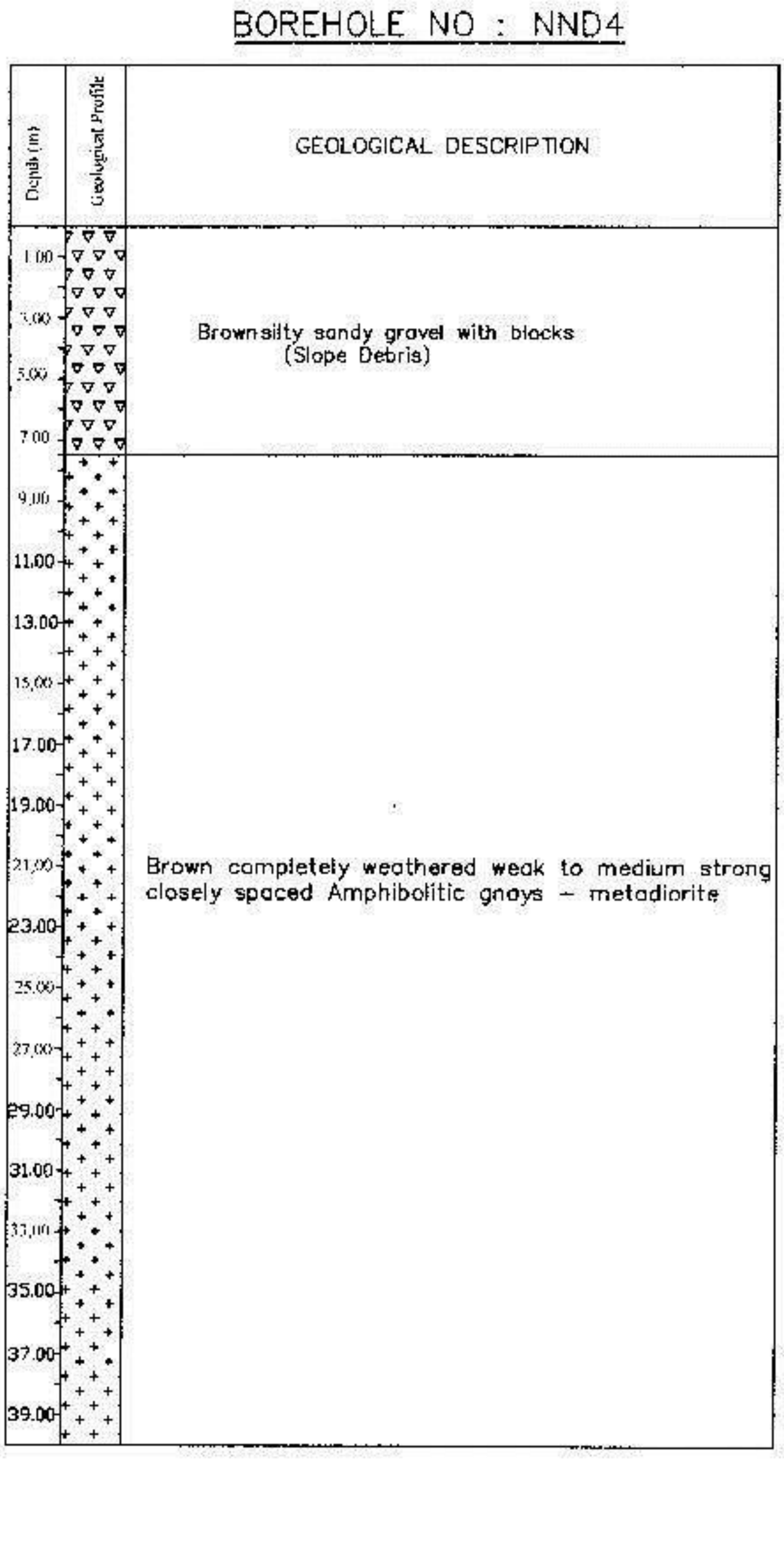


Figure 5.4 Borehole logs (NNB4 and NND4) [21]

Soil and rock types encountered along the motorway alignment between KM 15+060 to 15+187, their in-situ and laboratory characteristics and engineering properties are given in table 5.1 and 5.2.

Table 5.1 Soil Parameters for slope debris – soil type of Asarsuyu Valley

Natural Unit Weight	γ_n = 18,0 kN/m^3
Internal Friction Angle	$\phi = 30^0$
Deformation Modulus	E_s = 20 000 kN/m^2 E_{dyn} = 50 000 kN/m^2
Modulus of lateral soil reaction (Figure 4.22)	K_h = 28 000 kN/m^3

Table 5.2 Soil parameters for amphibolite and metadiorite soil type of Asarsuyu Valley

Natural Unit Weight	γ_n = 22,0 kN/m^3
Internal Friction Angle	$\phi = 38^0$
Cohesion	c = 30 kPa
Deformation Modulus	E_s = 100 000 kN/m^2 E_{dyn} = 200 000 kN/m^2
Point Load Index	I_s = 0,20 MPa
Uniaxial Compression Strength	$q_u = 24 \times 0,20 = 4,8$ MPa
Modulus of lateral soil reaction (Figure 4.22)	K_h = 65 000 kN/m^3

5.1.2 SOIL NAIL DESIGN FOR EXCAVATION BETWEEN KM 15+060 – KM 15+187 AT NORTHERN SLOPE

The soil nailed wall is designed using the TALREN 97 program (Appendix 1), which is currently the standard procedure used in France to design soil nailed walls. This method is an at limit equilibrium slices analysis and is based on a multi-criteria method proposed by Schlosser [10]. The four failure criteria taken into account relate to the different failure modes of a soil nailed wall and to the different soil nail interactions. The method considers the following resistances: the tensile and bending strength of the inclusion, the shear strength of the soil, the maximum soil-inclusion lateral friction, and the maximum lateral earth pressure on the reinforcement.

The TALREN design method does not calculate the tensile forces or bending moments mobilized in each row of nails, but rather the design is generally made by considering the resistance of the nails in the stability of all construction phases and the constructed wall.

Soil-nailed cuts have been designed with a 4V/1H slope and a B=3,0 m wide berm is provided every 12.0 m excavation height. Relatively long nails with higher capacities are provided in nail design considering the excavation height and properties of the soil/rock formations that are encountered. A closer soil nail pattern has been designed for slope debris, whereby it is aimed to improve the shear strength of the composite material made of soil-cement mortar and steel nails. Nail lengths have been selected as L= 12,0 m – 16,0 m –24, 0 m with a nail diameter of 36,0 mm, drill hole diameter shall be 125 mm, while nail inclination of 15^0 is designed. For the upper part of the cut where slope debris is encountered in general, nails with St III quality steel (f_y = 420 MPa) shall be used with a nail pattern of S_v = 1,5 m, S_h =1,0m. On the other hand, the lower part of cut slope, where there is weathered rock, high quality steel bars UTS 1080/1230 (f_y=1080 MPa) are to be used with a pattern of S_v= 2,0 m, S_h =2,0 m (Figure 5.6).

Here we are looking at the case of a soil nailed wall, a total height of 27 m. The typical features of the soils are shown in table 5.3.

Table 5.3 Values of the parameters in the design of a soil nailed wall between KM 15+060 – KM 15+187

		SOIL TYPE	
	DESCRIPTION	SLOPE DEBRIS	AMPHIBOLITE AND METADIORITE
Unit Skin Friction (F_1)	At the project design stage, the characteristic value for the soil unit skin friction (F_1) will be determined based on the charts or from in-situ pull-out tests. (Table 3.1, 3.2 and 3.3)	F_1 = 70 kN/m^2	F_1 = 200 kN/m^2
Limit Pressure of the Soil (p_1)	The French National Research Project CLOUTERRE (1991) provided a data base to suggest preliminary design charts that yield correlations between the unit skin friction (F_1) and the pressure limit p_1 obtained with the pressuremeter in different types of soils for both driven nails and gravity grouted nails. (Figure 4.15 to 4.19)	p_1 = 800 kN/m^2	p_1 = 1700 kN/m^2

The characteristic nail strength values (R_n, R_c and M) will be calculated on the basis of the guaranteed elastic limit of the steel where the nails include a metal reinforcing bar.

During the design we made assumption that the nails work only in tension. Under these circumstances, there remain only the nail failure criterion and the soil-nail skin friction criterion. This can be acceptable if nails have a low moment of inertia (small cross-sections like in driven nails), are nearly horizontal (inclination to the horizontal lower than 20 degrees) and if there is not any surcharge on top of the wall [6]. The nails tension forces with St III quality and UTS 1080/1230 high quality steel bars are expressed in table 5.4 and 5.5.

Table 5.4 The tension force with St III quality steel

EI	1,73 t m^2
L_o (Equation 4.15)	0,512 m
L_a	1,536 m
R_n	420 kN
Γ_{mRn}	1,15
T_n (Equation 4.19)	301,44 kN

Table 5.5 The tension force with UTS 1080/1230 quality steel

EI	1,73 t m^2
L_o (Equation 4.15)	0,415 m
L_a	1,244 m
R_n	1080 kN
Γ_{mRn}	1,15
T_n (Equation 4.19)	698 kN

5.1.2.1 Static Loading Case

Minimum safety factors of Γ_{min} = 0,55 < 1,0 has been determined for static loading case when there is no soil nails (Fig 5.5).

Soil nail design for excavation in northern slopes between KM 15+060 – 15+187 is shown on figure 5.6. According to this soil nail design, Γ_{min} is 1.14, 1.18, 1.06 > 1.0 (Fig. 5.7, 5.8, 5.9).

5.1.2.2 Seismic Loading Case

Minimum safety factor is Γ_{min} = 1.03 >1.0 for earthquake loading case with a horizontal ground acceleration coefficient of a_h = 0,27 (Fig 5.10). For shallow failure surfaces in earthquake loading with a coefficient of horizontal acceleration of a_h = 0,49, a minimum safety factor Γ_{min} = 1.08 > 1.0 is determined (Fig 5.11).

The structure will be stable if Γ_{min} is higher than 1,0 [6]. As it may seen from table 5.6, soil nailed cut slopes with a 4V/1H inclination shall stay stable between KM 15+060 – 15+187.

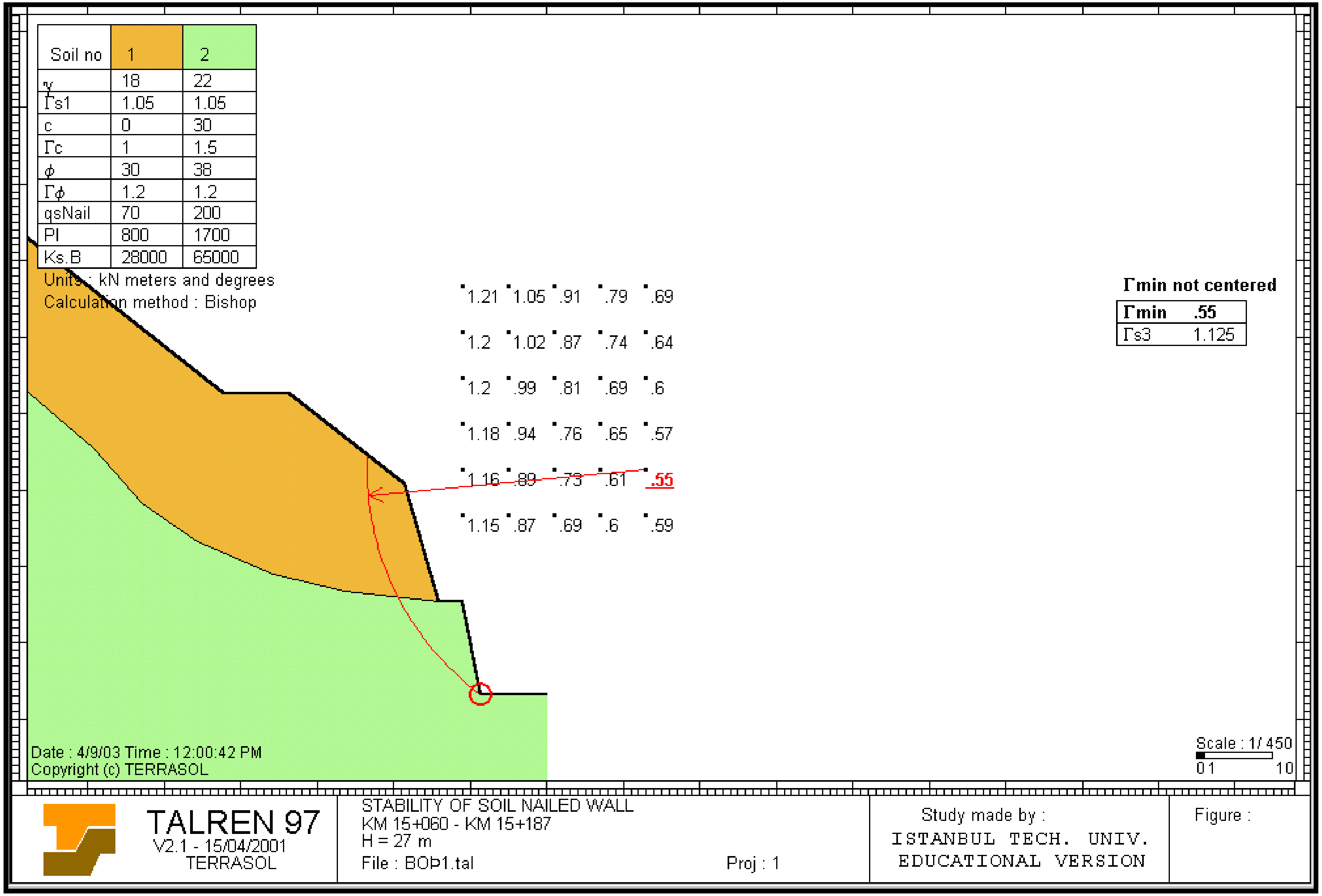


Figure 5.5 Stability analysis when there is no soil nails

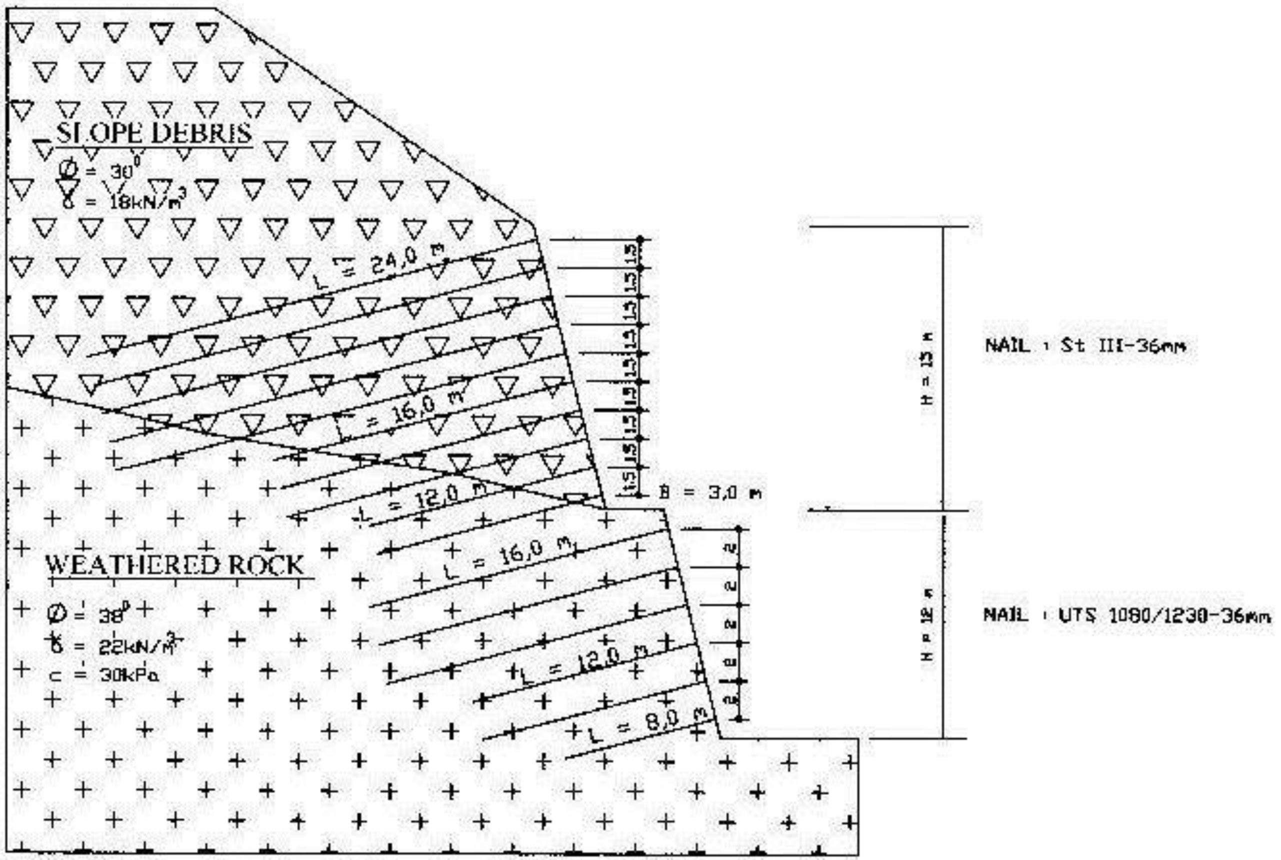


Figure 5.6 Soil nail design for excavation in Northern Slopes between KM 15 + 060 - KM 15 + 187 (H=27m)

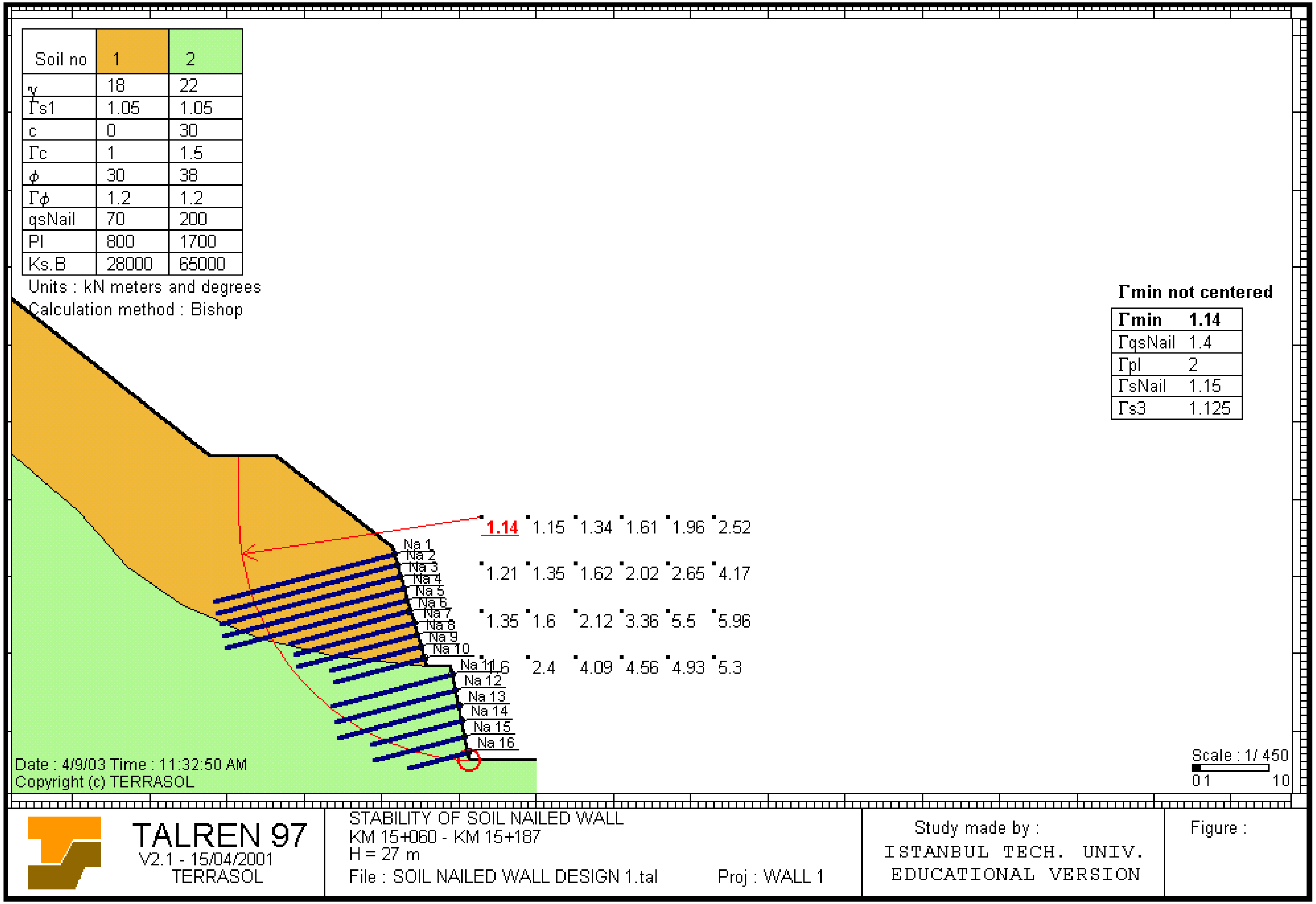


Figure 5.7 First static design and stability analysis of the soil nailed wall with TALREN 97

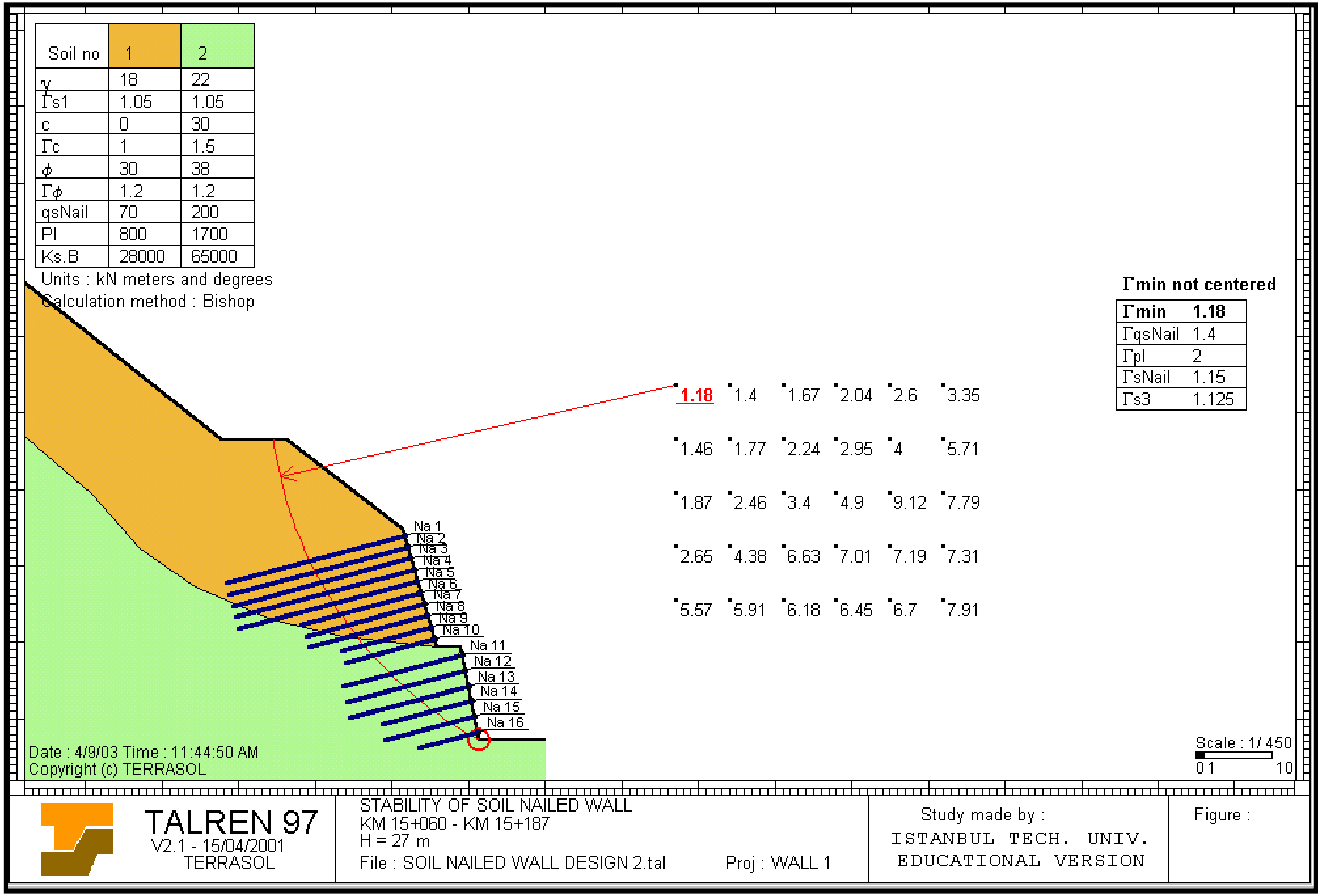


Figure 5.8 Second static design and stability analysis of the soil nailed wall with TALREN 97

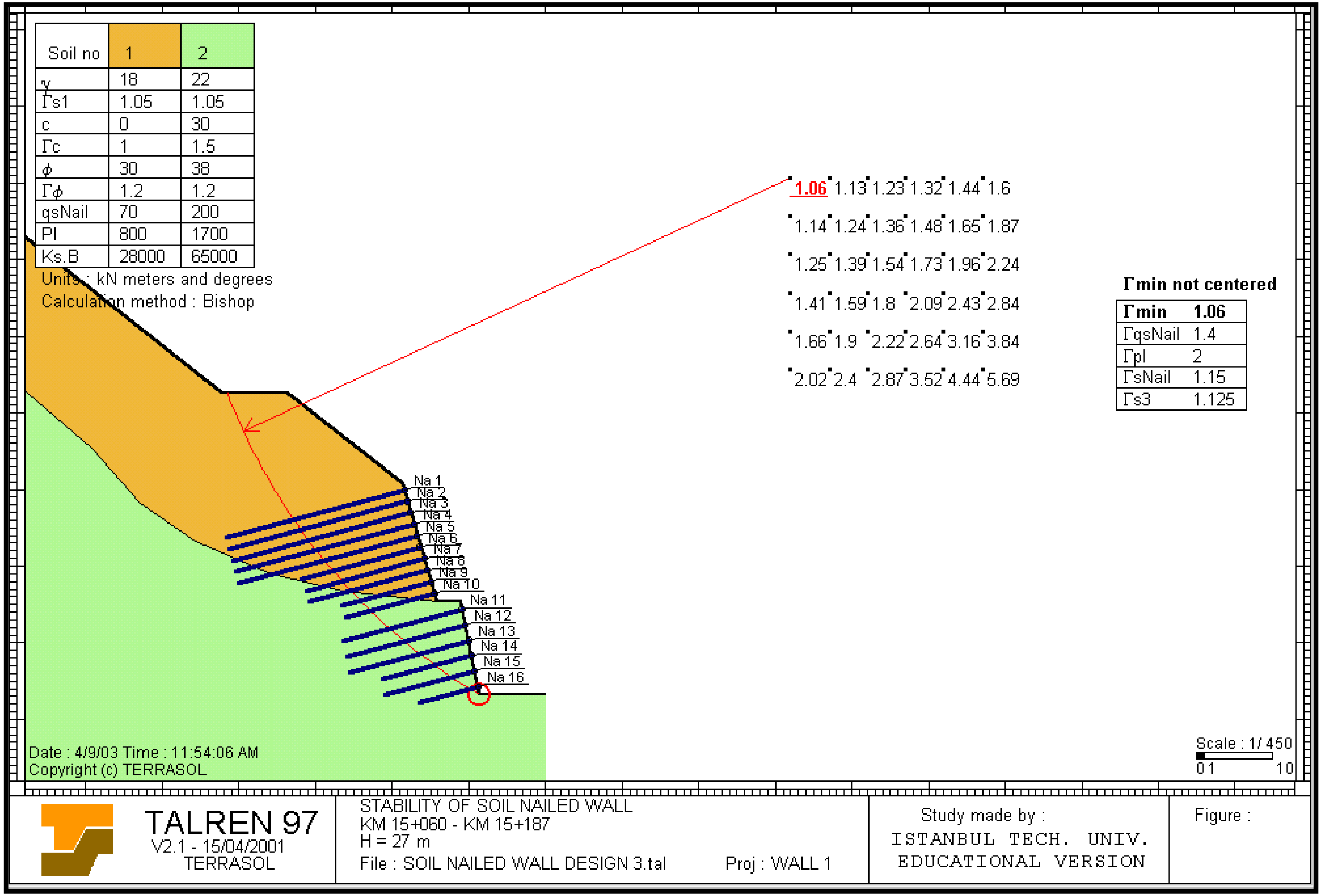


Figure 5.9 Third static design and stability analysis of the soil nailed wall with TALREN 97

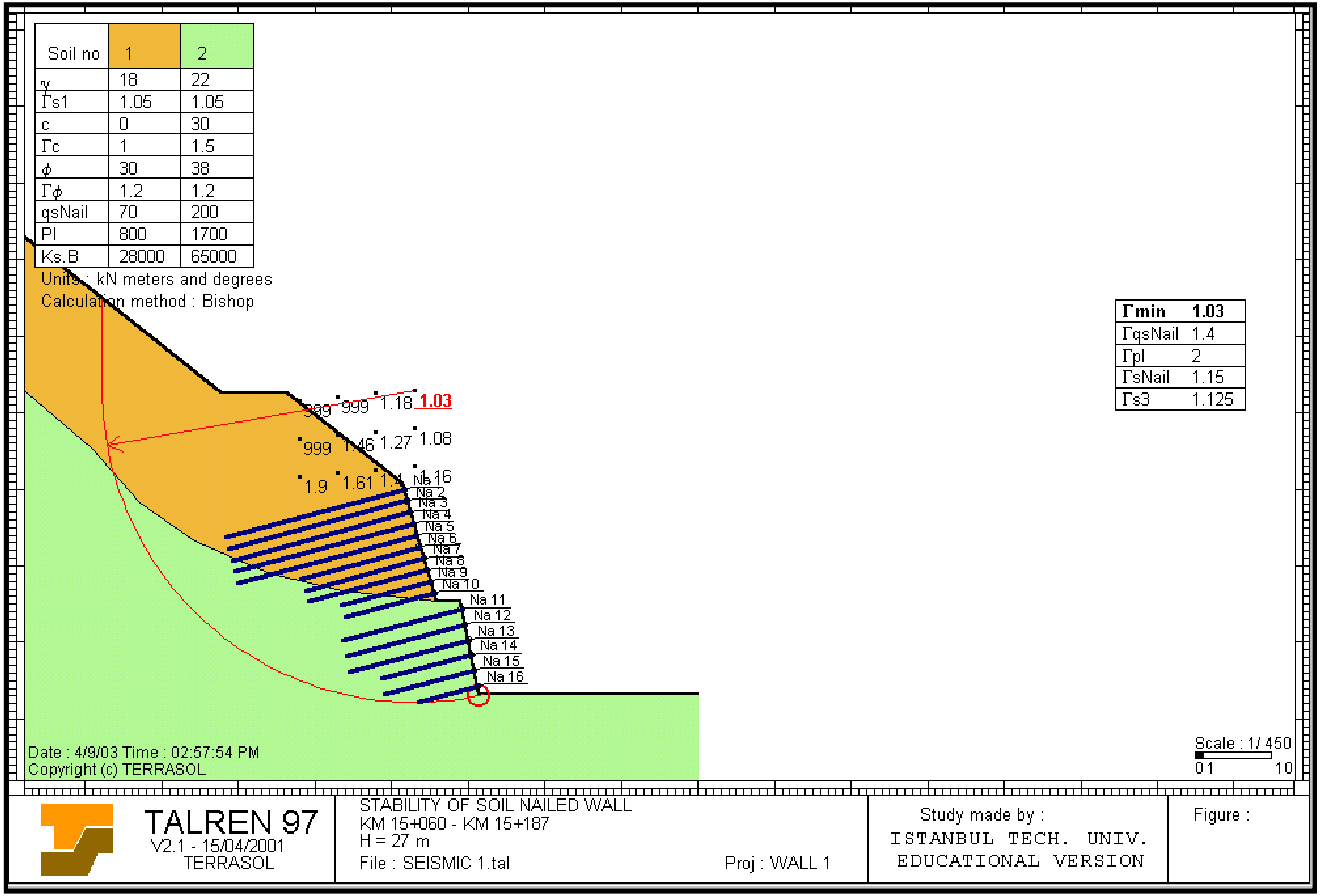


Figure 5.10 Seismic design and stability analysis of the soil nailed wall with TALREN 97 (For deep failure surface)

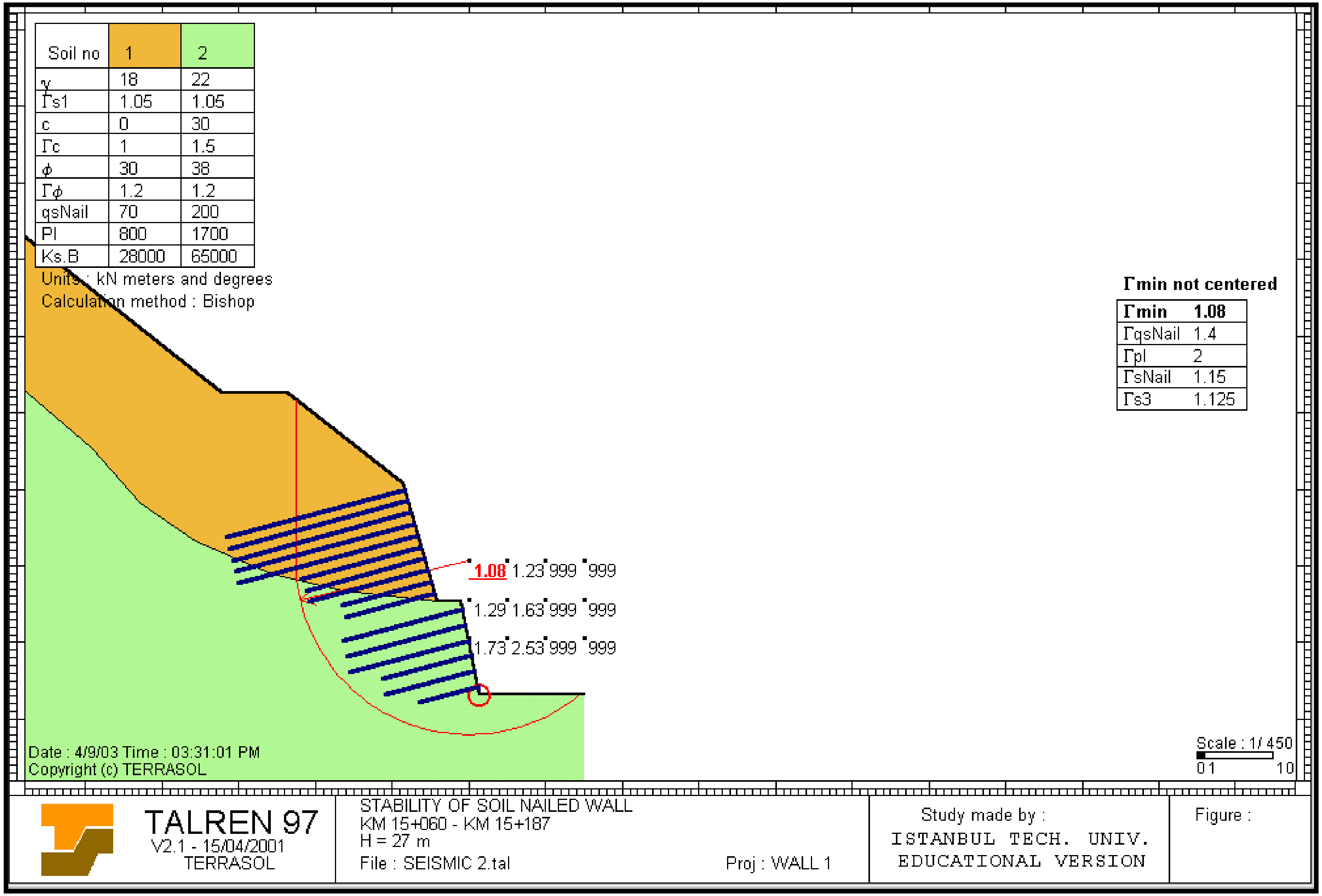


Figure 5.11 Seismic design and stability analysis of the soil nailed wall with TALREN 97 (For shallow failure surface)

Table 5.6 Soil nail design for cut slope between KM 15+060 – KM 15+187

	SOIL NAIL DESIGN					
Excavation Height (m)	Over the Berm	Below the Berm	Factor of Safety (Γ_{min})			Related Figures
			Static	Seismic		
				Deep	Shallow	
H = 27 m	S_v = 1,5 m S_h = 1,0 m ST-III D_a = 36 mm D_c = 125 mm L = 12 m-16 m- 24,0 m	S_v = 2,0 m S_h = 2,0 m UTS 1080/1230 D_a = 36 mm D_c = 125 mm L = 8,0 m-12,0 m- 24,0 m	1,14 1,18 1,06 ≅ 1,13 > 1,0	1,03 > 1,0	1,08 > 1,0	<u>For Static Loading</u> Figure 5.7 – Figure 5.9 <u>For Seismic Loading</u> Figure 5.10, Figure 5.11

6.CONCLUSIONS AND RECOMMENDATIONS

To date, soil nailing has been primarily used for temporary retaining structures. This is mainly due to the engineering concerns with regard to durability of metallic inclusions in the ground and shortcomings of facing technology. In recent years, technological developments overcome these limitations.

Today the technique of soil nailing is far spread and advanced in Germany, France, Great Britain, Japan and the United States.

Predictions of the stability of soil nailed walls depend on the slope of the face, the capacity of the nails, the length of the nails, the number of nails and their inclination to the horizontal, the strength characteristics of the soil, and the methods adopted to analyze the soil nailed wall.

The number of rows and of nails in a wall have a great effect on the failure surface location and the factor of safety. The more rows of nails in a vertical section of wall, the more difficult it is for the failure surface to avoid the nails. Additionally, inclined nails make it more difficult for the failure surface to avoid intersection rows of nails.

In this study, the soil nailing technique and design of soil nailed retaining structures are examined. A computer program, TALREN 97, has been applied to evaluate the stability of reinforced slopes. TALREN 97, developed by TERRASOL, is a stability analysis program for geotechnical structures along potential failure surfaces. The program considers hydraulic and seismic data, in addition to various types of soil inclusions (nail, anchor, brace, reinforcing strip, geotextile, pile, micropile, sheetpile, etc.).

The TALREN design method does not calculate the tensile forces or bending moments mobilized in each row of nails, but rather the design is generally made by considering the resistance of the nails in the stability of all construction phases and the constructed wall.

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[25] http://www.contechsystems.com/Projects/B_I15/hex2.jpg (March 15, 2003)

[26] http://www.yenter.net/soil_nail_launcher.htm (March 15, 2003)

[27] http://tc17.poly.edu/sn.htm (March 15, 2003)

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[29] http://www.stormwaterpro.org/ec_0109_soil.html (March 15, 2003)

[30] http://www.jhri.jhnet.go.jp/Research/geot-e/dk4.htm (March 15, 2003)

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APPENDIX I

Talren 97 - Computer Program For The Stability Analysis of Geotechnical Structures;

A.1 Calculation Method

A.1.1 General Principles

A.1.2 Geometry

A.1.3 Failure Surfaces

A.1.4 Hydraulic Conditions

A.1.5 Surcharges

A.1.6 Seismic Loadings

A.1.7 Forces in the Nails

TALREN 97 – COMPUTER PROGRAM FOR THE STABILITY ANALYSIS OF GEOTECHNICAL STRUCTURES

TALREN 97, developed by TERRASOL, is a stability analysis program for geotechnical structures along potential failure surfaces. The program considers hydraulic and seismic data, in addition to various types of soil inclusions (nail, anchor, brace, reinforcing strip, geotextile, pile, micropile, sheetpile, etc.). Its development was carried out concurrently with experimental research on soil-inclusion interaction and the design of actual structures [18].

A.1 CALCULATION METHOD

A.1.1 General Principles

The program allows determination of the stability of a geotechnical structure (excavation, fill, etc.), with or without reinforcement (nails, anchors, reinforcing strips, braces, piles, etc.). TALREN 97 is based on classical slope stability methods considering a failure surface at limit equilibrium. The validity of these methods has been proven for nearly 40 years by more than a thousand actual structures. The equilibrium of the active soil mass, located between the slope surface and a circular, polygonal or any shape failure surface, is analyzed by conventional methods, i.e.: Fellenius or Bishop slice methods, or the Perturbation method [32].

In these methods, the soil is divided into discrete or elemental vertical slices, for which the static equilibrium is analyzed (see figure A.1). The safety factor Γ, assumed constant along the failure surface, is defined as the ratio of the maximum shear strength τ_{max} to the mobilized shear stress τ along the failure surface. The system equilibrium of the soil is determined using the reduced strength parameters c / Γ_c and $\tan \phi / \Gamma_\phi$ (c is the cohesion and ϕ is the internal friction angle).

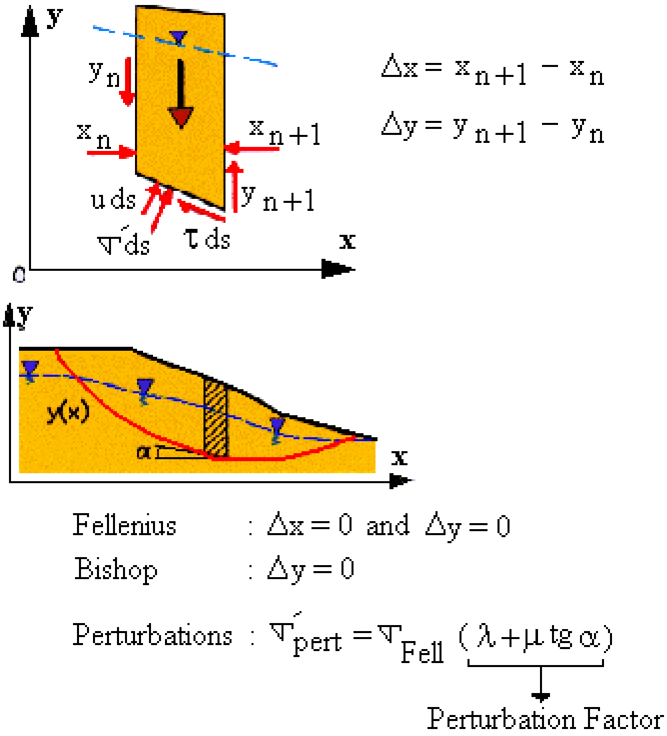

Figure A.1 Equilibrium of a slice of soil [32]

A.1.2 Geometry

TALREN 97 accepts all possible slope and soil profile geometries (Figure A.2). The geometry is defined by points and segments, using open or closed polygonal lines. This allows the definition of complex geometries.

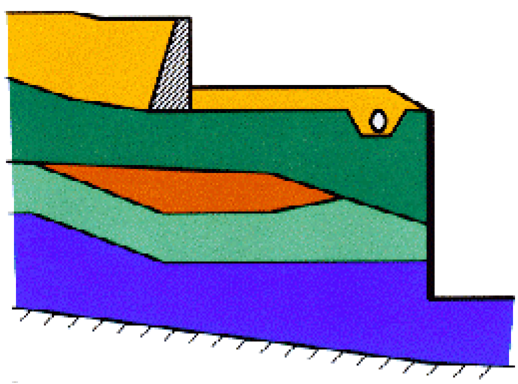

Figure A.2 Example of a complex geometry [32]

A.1.3 Failure surfaces

The program can analyze circular and all types of polygonal failure surfaces (Figure A.3).

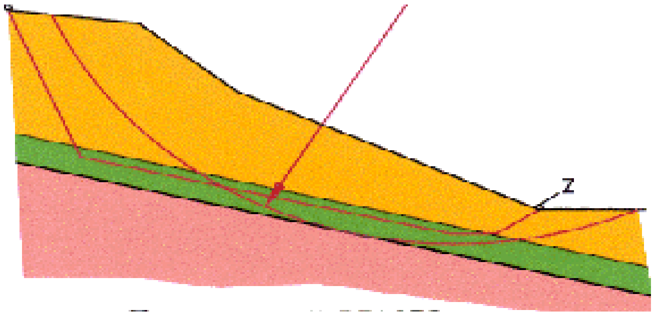

Figure A.3 Failure Surfaces [32]

A.1.4 Hydraulic Conditions

Four possible options exist for computing pore pressures along the failure surfaces (Figures A.4a, A.4b, and A.4c):

- a phreatic surface geometrically defined by points, with the possibility of introducing seepage by imposing the equipotential line at each point;
- pore pressures given at every point along a non-circular failure surface;
- pore pressures defined at every node of a triangular mesh, whose values were obtained, for example, from finite element seepage analysis;

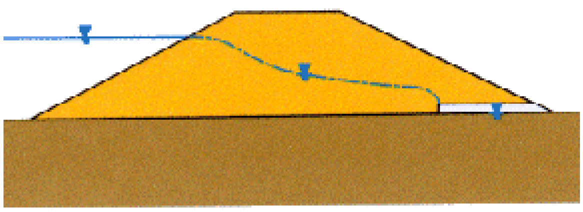

Figure A.4a Hydraulic conditions defined by the top of a water table [32]

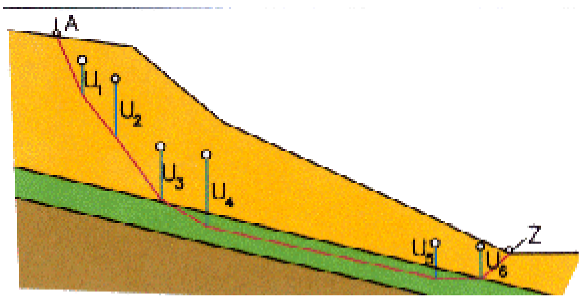

Figure A.4b Hydraulic conditions defined along a non-circular failure surfaces[32]

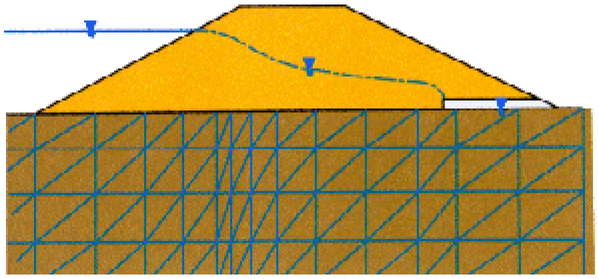

Figure A.4c Hydraulic conditions defined at the nodes of a triangular mesh [32]

The program can also treat external water tables by considering the horizontal forces, equal to the hydrostatic pressure applied at the endpoints of the failure surface, in the global equilibrium (Figure A.5).

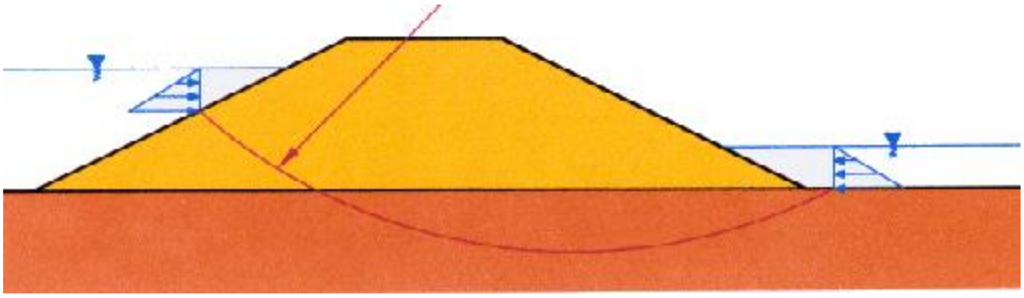

Figure A.5 Hydrostatic pressure of external water at the exit points of the failure surface [32]

A.1.5 Surcharges

Three types of surcharges can be applied (Figure A.6):

- Vertical distributed surcharges, which increase the weight of each soil slice on which they are applied (discretization of the failure surface) in proportion to the slice thickness;
- Line loads which induce additional soil stresses along the failure surface. This increase is taken into account by considering the shear stress ($\Delta\tau$) and normal stress ($\Delta\sigma$) increments in the equilibrium equations;
- Additional moments which are added to or subtracted from the driving moment. For circular failure surfaces, additional moments can only be considered when the Fellenius or Bishop analysis methods are used.

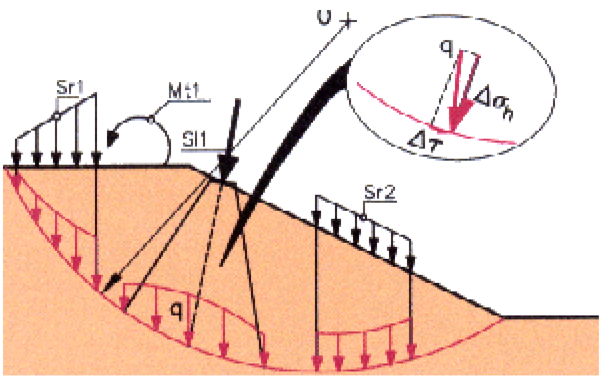

Figure A.6 Application of surcharges [32]

A.1.6 Seismic Loadings

Seismic loads are treated with a pseudo-static approach by introducing the forces associated with the horizontal and/or vertical accelerations (Figure A.7). One should note that:

- the vertical coefficient is applied to the soil, the surcharges and the water;
- the horizontal coefficient is only applied to the soil and the water located within the soil.

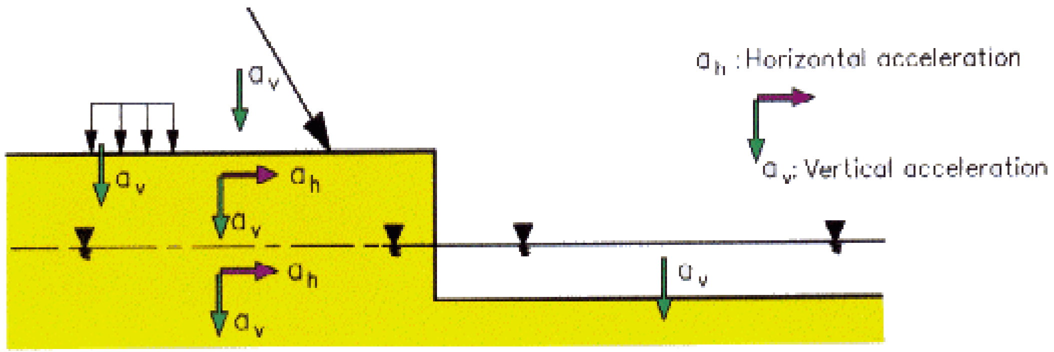

Figure A.7 Unit forces associated with seismic accelerations [32]

A.1.7 Forces in the Nails

When nails are introduced in the soil, the mobilized forces in these elements, at the intersection with the failure surfaces, should be considered in the static equilibrium.

The forces taken into account are:

- the axial force
- the shear force and bending moment for nails working in coupled tension/shear or pure shear

These forces depend on the mechanical characteristics of the soil since they are mobilized by soil/inclusion interaction (lateral friction, lateral pressure between the soil and the nails).

APPENDIX II

Data and Results output for file;

- Soil Nailed Wall Design 1
- Soil Nailed Wall Design 2
- Soil Nailed Wall Design 3
- Seismic 1
- Seismic 2

Fichier de données : c:\SOIL NAILED WALL DESIGN 1.rel
Date :Apr 9 2003
Time :12h 3mn 44s

Name of data file :c:\SOIL NAILED WALL DESIGN 1.tal

```
!-----------------------------------------------------!
!                                                     !
!                TALREN 97 program                    !
!                                                     !
!      (TALus RENForces = reinforced slopes)          !
!                                                     !
!                       -o-                           !
!                                                     !
!            TALREN 97 1.1   of   02/15/98            !
!                                                     !
!   Computer program for the stability analysis       !
!              of reinforced slopes                   !
!                                                     !
!                                                     !
!                       -o-                           !
!                                                     !
!         Copyright (c) 1981 TALREN - TERRASOL        !
!                                                     !
!-----------------------------------------------------!
! !-----------!       Developed  by TERRASOL          !
! !           !                                       !
! !-!  /----! !          IMMEUBLE HELIOS              !
! ! !  / /---!          72 Avenue PASTEUR             !
! !  !--/ /   !   93108 MONTREUIL cedex - FRANCE      !
! !-----/  /-!                                        !
! !       /        Telephone : 00 33 1 49 88 24 42    !
! !-------/                                           !
!-----------------------------------------------------!
```

Project number ..:WALL 1
Project location :
Title ....:STABILITY OF SOIL NAILED WALL

Comment(s) :
- KM 15+060 - KM 15+187
- H = 27 m

```
*****************************************************************************************************
*                                                                                                   *
*   DATA  -  DATA  -  DATA  -  DATA  -  DATA  -  DATA  -  DATA  -  DATA  -  DATA                    *
*                                                                                                   *
*****************************************************************************************************
```

* ANALYSIS METHOD:BISHOP
****

- Initial value of F: 1.00
- No zoning for hydraulic data
- Number of subdivisions for the failure surface: 49

* FAILURE SURFACE: CIRCULAR
****

- XO = 63.00 YO = 22.00 DX = 6.00 DY = 6.00 AX = .0 AY = .0 NX = 6 NY = 4
- Maximum number for circles for the calculation 10
- Radius increment : 5.0
- Point of passage for the first circle: X = 61.420 Y = 8.580
- Minimum X value for the second intersection point of the failure surface with the slope: .00

* GEOMETRICAL DATA DEFINED BY POINTS AND SEGMENTS
****

POINT	X	Y
1	.00	70.00
2	27.84	47.87
3	36.42	47.87
4	51.42	36.10
5	56.00	20.71
6	59.00	20.71
7	61.42	8.58
8	70.00	8.58
9	.00	50.00
10	10.71	40.71
11	17.13	33.58
12	24.29	26.58
13	34.26	24.29
14	43.58	22.13

SEG No	ENDPT 1	ENDPT 2	UNDER SOIL
1	1	2	1
2	2	3	1
3	3	4	1
4	4	5	1
5	5	6	2
6	6	7	2
7	7	8	2
8	9	10	2
9	10	11	2
10	11	12	2
11	12	13	2
12	13	14	2
13	14	5	2

SLOPE SEGMENT NO
1
2
3
4
5
6
7

* SOIL CHARACTERISTICS

SOIL	GAMMA	FS1	COHES c	Dc/z	Fc	PHI	Fphi	RU	QS	PL	KSB
1	18.0	1.05	.0	.00	1.00	30.0	1.20	.00	70.0	800.0	28000.0
2	22.0	1.05	30.0	.00	1.50	38.0	1.20	.00	200.0	1700.0	65000.0

* REINFORCEMENT DATA

No.	TYPE	STRENGTH	SPAC.	X	Y	L	ANG	LB	DA	RAL	REQ	IND	SHEAR	L MIN	MMAX	EI	CA	IENC
1	NAIL	301.4	1.00	51.61	35.35	24.0	15.0	2.00	20.0	.0	.063	1	.0	.00	.0	.0	.0	0
2	NAIL	301.4	1.00	51.98	33.85	24.0	15.0	2.00	20.0	.0	.063	1	.0	.00	.0	.0	.0	0
3	NAIL	301.4	1.00	52.40	32.35	24.0	15.0	2.00	20.0	.0	.063	1	.0	.00	.0	.0	.0	0
4	NAIL	301.4	1.00	52.85	30.85	24.0	15.0	2.00	20.0	.0	.063	1	.0	.00	.0	.0	.0	0
5	NAIL	301.4	1.00	53.18	29.35	24.0	15.0	2.00	20.0	.0	.063	1	.0	.00	.0	.0	.0	0
6	NAIL	301.4	1.00	53.60	27.85	16.0	15.0	2.00	20.0	.0	.063	1	.0	.00	.0	.0	.0	0
7	NAIL	301.4	1.00	54.20	26.35	16.0	15.0	2.00	20.0	.0	.063	1	.0	.00	.0	.0	.0	0
8	NAIL	301.4	1.00	54.60	24.85	16.0	15.0	2.00	20.0	.0	.063	1	.0	.00	.0	.0	.0	0
9	NAIL	301.4	1.00	55.00	23.35	12.0	15.0	2.00	20.0	.0	.063	1	.0	.00	.0	.0	.0	0
10	NAIL	301.4	1.00	55.50	21.85	12.0	15.0	2.00	20.0	.0	.063	1	.0	.00	.0	.0	.0	0
11	NAIL	698.0	2.00	59.10	19.71	16.0	15.0	2.00	20.0	.0	.063	1	.0	.00	.0	.0	.0	0
12	NAIL	698.0	2.00	59.50	17.71	16.0	15.0	2.00	20.0	.0	.063	1	.0	.00	.0	.0	.0	0
13	NAIL	698.0	2.00	60.00	15.71	16.0	15.0	2.00	20.0	.0	.063	1	.0	.00	.0	.0	.0	0
14	NAIL	698.0	2.00	60.40	13.71	12.0	15.0	2.00	20.0	.0	.063	1	.0	.00	.0	.0	.0	0
15	NAIL	698.0	2.00	60.75	11.71	12.0	15.0	2.00	20.0	.0	.063	1	.0	.00	.0	.0	.0	0
16	NAIL	698.0	2.00	61.30	9.71	8.0	15.0	2.00	20.0	.0	.063	1	.0	.00	.0	.0	.0	0

- CALCULATION TYPE: - For NAILS: IND = 1 Tension calculated / Shear imposed
IND = 2 Zero tension / Shear calculated
IND = 3 Tension and Shear calculated

- For ANCHORS: IND = 1 Pull-out resistance as a function of the length beyond the failure surface
IND = 2 Pull-out resistance either total or zero depending on the position of the fictitious a

- Taking into account the portion of the reinforcement inside or outside the failure surface

	SHEAR	TENSION
. IENC = 0 :	External/Internal	External
. IENC = 1 :	External	External
. IENC = 2 :	External/Internal	External/Internal
. IENC = 3 :	External	External/internal

- The increase in normal stresses along the failure surface induced by the reinforcement is calculated by the program

* PARTIAL SAFETY FACTORS FOR THE STRENGTH PARAMETERS

- Soil/inclusion lateral friction
. Nail....................FqsNa : 1.400
. Anchor ....FqsAn : 1.000
. Reinforcing strip......FqsRS : 1.000
- Limit pressure of soils..................Fpl : 2.000
- Intrinsic strength of the inclusions
. Nail ....FaNai : 1.150
. Anchor ....FaAnc : 1.000
. Reinforcing strip......FaRS : 1.000
. Brace ....FaBra : 1.000
- Calculation method ....Fs3 : 1.125

Fichier de donnees : c:\SOIL NAILED WALL DESIGN 1.re2
Date :Apr 9 2003
Time :13h 12mn 30s

Name of data file :c:\SOIL NAILED WALL DESIGN 1.tal

RESULTS - RESULTS - RESULTS - RESULTS - RESULTS - RESULTS - RESULTS - RESULTS - RESULTS

Indicators (*) - INT : There are other intersections between the failure surface and the slope
- For the safety factors
it indicates that convergence was not obtained

METHOD (analysis method): 1 - FELLENIUS
2 - BISHOF
3 - PERTURBATIONS
- The overturning moments are computed taking into account the defined loads

- LIMIT FORCES IN THE REINFORCEMENT :
. TENSION
by the STEEL STRENGTH if ITR = 1
by qs if ITR = 2
. SHEAR
Long inclusion/high bending stiffness if ISHR = 1
Long inclusion/low bending stiffness if ISHR = 2
Short inclusion/high bending stiffness if ISHR = 3
Short inclusion/low bending stiffness if ISHR = 4
limited by TR/2 if ISHR = 5

- INDICATOR FOR TAKING INTO ACCOUNT THE FORCES IN THE REINFORCEMENT
(Portion located inside or outside the failure surface)
TENSION SHEAR
IPTR = 0 IPSH = 0 No calculated forces
IPTR = 1 IPSH = 1 Calculation along the external portion imposed
IPTR = 2 IPSH = 2 Calculation along the outside = min(inside
outside)
IPTR = 3 IPSH = 3 Calculation along the inside = min(inside
outside)
IPSH = 4 Long inclusion
calculated along both sides
IPSH = 5 SHEAR is imposed

- UNITS : kN
m and degrees

XO	YO	RADIUS	OVER MOMENT	F-SOIL	F-SURCH	F-TOTAL	METH	INT
63.00	22.00	13.51	32959.02	.92	.92	1.60	2	*
63.00	22.00	503.51	32959.02	999.00	999.00	999.00	2	
69.00	22.00	15.41	17747.89	.76	.76	2.40	2	*
69.00	22.00	505.41	17747.89	999.00	999.00	999.00	2	
75.00	22.00	19.09	12419.04	.71	.71	4.09	2	*
75.00	22.00	509.09	12419.04	999.00	999.00	999.00	2	
81.00	22.00	23.73	10129.59	.78	.76	4.56	2	*
81.00	22.00	513.73	10129.59	999.00	999.00	999.00	2	
87.00	22.00	28.88	7839.81	1.01	1.01	4.93	2	*
87.00	22.00	518.88	7839.81	999.00	999.00	999.00	2	
93.00	22.00	34.31	5550.86	1.55	1.55	5.30	2	*
93.00	22.00	524.31	5550.86	999.00	999.00	999.00	2	
63.00	28.00	19.48	83973.40	.93	.93	1.35	2	*
63.00	28.00	509.48	83973.40	999.00	999.00	999.00	2	
69.00	28.00	20.84	59288.51	.72	.72	1.60	2	*
69.00	28.00	510.84	59288.51	999.00	999.00	999.00	2	
75.00	28.00	23.69	39955.08	.65	.65	2.12	2	*
75.00	28.00	513.69	39955.08	999.00	999.00	999.00	2	
81.00	28.00	27.57	26549.09	.66	.66	3.36	2	*
81.00	28.00	517.57	26549.09	999.00	999.00	999.00	2	
87.00	28.00	32.11	19574.04	.73	.73	5.50	2	*
87.00	28.00	522.11	19574.04	999.00	999.00	999.00	2	
93.00	28.00	37.07	17016.34	.80	.80	5.96	2	*
93.00	28.00	527.07	17016.34	999.00	999.00	999.00	2	
63.00	34.00	25.46	157848.00	.96	.96	1.21	2	*
63.00	34.00	515.46	157848.00	999.00	999.00	999.00	2	

69.00	34.00	26.52	122062.00	.75	.75	1.35	2	*
69.00	34.00	516.52	122062.00	999.00	999.00	999.00	2	
75.00	34.00	28.82	94893.84	.63	.63	1.62	2	*
75.00	34.00	518.82	94893.84	999.00	999.00	999.00	2	
81.00	34.00	32.08	72743.15	.56	.56	2.02	2	*
81.00	34.00	522.08	72743.15	999.00	999.00	999.00	2	
87.00	34.00	36.06	53664.71	.57	.57	2.65	2	*
87.00	34.00	526.06	53664.71	999.00	999.00	999.00	2	
93.00	34.00	40.54	37757.32	.63	.63	4.17	2	*
93.00	34.00	530.54	37757.32	999.00	999.00	999.00	2	
63.00	40.00	31.45	251406.90	.99	.99	1.14	2	*
63.00	40.00	521.45	251406.90	999.00	999.00	999.00	2	
69.00	40.00	32.32	206737.10	.79	.79	1.15	2	*
69.00	40.00	522.32	206737.10	999.00	999.00	999.00	2	
75.00	40.00	34.22	168167.60	.66	.66	1.34	2	*
75.00	40.00	524.22	168167.60	999.00	999.00	999.00	2	
81.00	40.00	37.02	137398.20	.57	.57	1.61	2	*
81.00	40.00	527.02	137398.20	999.00	999.00	999.00	2	
87.00	40.00	40.51	111842.60	.52	.52	1.96	2	*
87.00	40.00	530.51	111842.60	999.00	999.00	999.00	2	
93.00	40.00	44.54	89579.72	.51	.51	2.52	2	*
93.00	40.00	534.54	89579.72	999.00	999.00	999.00	2	

Fichier de données : c:\SOIL NAILED WALL DESIGN 2.rel
Date :Apr 9 2003
Time :13h 6mn 29s

Name of data file :c:\SOIL NAILED WALL DESIGN 2.tal

```
!-----------------------------------------------!
!                                               !
!              TALREN 97 program                !
!                                               !
!   (TALus RENForces = reinforced slopes)       !
!                                               !
!                     -o-                       !
!                                               !
!          TALREN 97 1.1  of  02/15/98          !
!                                               !
!  Computer program for the stability analysis  !
!            of reinforced slopes               !
!                                               !
!                     -o-                       !
!                                               !
!     Copyright (c) 1981 TALREN - TERRASOL      !
!-----------------------------------------------!
! !-----------!        Developed  by TERRASOL   !
! !           !                                 !
! !-!    /----!        IMMEUBLE HELIOS          !
!   !   / /---!        72 Avenue PASTEUR        !
!   !--/ /    !   93108 MONTREUIL cedex - FRANCE!
! !-----/   /-!                                 !
! !        /    Telephone : 00 33 1 49 88 24 42 !
! !-------/                                     !
!-----------------------------------------------!
```

Project number ...:WALL 1
Project location :
Title ....:STABILITY OF SOIL NAILED WALL

Comment(s) :
- KM 15+060 - KM 15+187
- H = 27 m

DATA - DATA - DATA - DATA - DATA - DATA - DATA - DATA - DATA

* ANALYSIS METHOD:BISHOP

- Initial value of F: 1.00
- No zoning for hydraulic data
- Number of subdivisions for the failure surface: 49

* FAILURE SURFACE: CIRCULAR

- XO = 87.00 YO = 27.00 DX = 7.00 DY = 7.00 AX = .0 AY = .0 NX = 6 NY = 5
- Maximum number for circles for the calculation 10
- Radius increment : 5.0
- Point of passage for the first circle: X = 61.420 Y = 8.580
- Minimum X value for the second intersection point of the failure surface with the slope: .00

* GEOMETRICAL DATA DEFINED BY POINTS AND SEGMENTS

POINT	X	Y
1	.00	70.00
2	27.84	47.87
3	36.42	47.87
4	51.42	36.10
5	56.00	20.71
6	59.00	20.71
7	61.42	8.58
8	70.00	8.58
9	.00	50.00
10	10.71	40.71
11	17.13	33.58
12	24.29	28.58
13	34.26	24.29
14	43.58	22.13

SEG No	ENDPT 1	ENDPT 2	UNDER SOIL
1	1	2	1
2	2	3	1
3	3	4	1
4	4	5	1
5	5	6	2
6	6	7	2
7	7	8	2
8	9	10	2
9	10	11	2
10	11	12	2
11	12	13	2
12	13	14	2
13	14	5	2

SLOPE SEGMENT NO
1
2
3
4
5
6
7

* SOIL CHARACTERISTICS

SOIL	GAMMA	FS1	COHES c	Dc/z	Fc	PHI	Fphi	RU	QS	PL	KSB
1	18.0	1.05	.0	.00	1.00	30.0	1.20	.00	70.0	800.0	28000.0
2	22.0	1.05	30.0	.00	1.50	38.0	1.20	.00	200.0	1700.0	65000.0

* REINFORCEMENT DATA

No.	TYPE	STRENGTH	SPAC.	X	Y	L	ANG	LB	DA	RAL	REQ	IND	SHEAR	L MIN	MMAX	EI	CA	IENC
1	NAIL	301.4	1.00	51.61	35.35	24.0	15.0	2.00	20.0	.0	.063	1	.0	.00	.0	.0	.0	0
2	NAIL	301.4	1.00	51.98	33.85	24.0	15.0	2.00	20.0	.0	.063	1	.0	.00	.0	.0	.0	0
3	NAIL	301.4	1.00	52.40	32.35	24.0	15.0	2.00	20.0	.0	.063	1	.0	.00	.0	.0	.0	0
4	NAIL	301.4	1.00	52.85	30.85	24.0	15.0	2.00	20.0	.0	.063	1	.0	.00	.0	.0	.0	0
5	NAIL	301.4	1.00	53.18	29.35	24.0	15.0	2.00	20.0	.0	.063	1	.0	.00	.0	.0	.0	0
6	NAIL	301.4	1.00	53.60	27.85	16.0	15.0	2.00	20.0	.0	.063	1	.0	.00	.0	.0	.0	0
7	NAIL	301.4	1.00	54.20	26.35	16.0	15.0	2.00	20.0	.0	.063	1	.0	.00	.0	.0	.0	0
8	NAIL	301.4	1.00	54.60	24.85	16.0	15.0	2.00	20.0	.0	.063	1	.0	.00	.0	.0	.0	0
9	NAIL	301.4	1.00	55.00	23.35	12.0	15.0	2.00	20.0	.0	.063	1	.0	.00	.0	.0	.0	0
10	NAIL	301.4	1.00	55.50	21.85	12.0	15.0	2.00	20.0	.0	.063	1	.0	.00	.0	.0	.0	0
11	NAIL	698.0	2.00	59.10	19.71	16.0	15.0	2.00	20.0	.0	.063	1	.0	.00	.0	.0	.0	0
12	NAIL	698.0	2.00	59.50	17.71	16.0	15.0	2.00	20.0	.0	.063	1	.0	.00	.0	.0	.0	0
13	NAIL	698.0	2.00	60.00	15.71	16.0	15.0	2.00	20.0	.0	.063	1	.0	.00	.0	.0	.0	0
14	NAIL	698.0	2.00	60.40	13.71	12.0	15.0	2.00	20.0	.0	.063	1	.0	.00	.0	.0	.0	0
15	NAIL	698.0	2.00	60.75	11.71	12.0	15.0	2.00	20.0	.0	.063	1	.0	.00	.0	.0	.0	0
16	NAIL	698.0	2.00	61.30	9.71	8.0	15.0	2.00	20.0	.0	.063	1	.0	.00	.0	.0	.0	0

- CALCULATION TYPE: - For NAILS: IND = 1 Tension calculated / Shear imposed
IND = 2 Zero tension / Shear calculated
IND = 3 Tension and Shear calculated

- For ANCHORS: IND = 1 Pull-out resistance as a function of the length beyond the failure surface
IND = 2 Pull-out resistance either total or zero depending on the position of the fictitious a

- Taking into account the portion of the reinforcement inside or outside the failure surface

	SHEAR	TENSION
. IENC = 0 :	External/Internal	External
. IENC = 1 :	External	External
. IENC = 2 :	External/Internal	External/Internal
. IENC = 3 :	External	External/Internal

- The increase in normal stresses along the failure surface induced by the reinforcement is calculated by the program

* PARTIAL SAFETY FACTORS FOR THE STRENGTH PARAMETERS

```
- Soil/inclusion lateral friction
. Nail...................FqsNa :  1.400
. Anchor ................FqsAn :  1.000
. Reinforcing strip......FqsRS :  1.000
- Limit pressure of soils..................Fpl   :  2.000
- Intrinsic strength of the inclusions
. Nail ..................FaNai :  1.150
. Anchor ................FaAnc :  1.000
. Reinforcing strip......FaRS  :  1.000
. Brace  ................FaBra :  1.000
- Calculation method .......................Fs3   :  1.125
```

Fichier de données : c:\SOIL NAILED WALL DESIGN 2.re2
Date : Apr 9 2003
Time : 13h 6mn 29s

Name of data file : c:\SOIL NAILED WALL DESIGN 2.tal

RESULTS - RESULTS - RESULTS - RESULTS - RESULTS - RESULTS - RESULTS - RESULTS - RESULTS

Indicators (*) - INT : There are other intersections between the failure surface and the slope
- For the safety factors
it indicates that convergence was not obtained

METHOD (analysis method): 1 - FELLENIUS
2 - BISHOP
3 - PERTURBATIONS
- The overturning moments are computed taking into account the defined loads

- LIMIT FORCES IN THE REINFORCEMENT :
. TENSION
by the STEEL STRENGTH if ITR = 1
by qs if ITR = 2
. SHEAR
Long inclusion/high bending stiffness if ISHR = 1
Long inclusion/low bending stiffness if ISHR = 2
Short inclusion/high bending stiffness if ISHR = 3
Short inclusion/low bending stiffness if ISHR = 4
limited by TR/2 if ISHR = 5

- INDICATOR FOR TAKING INTO ACCOUNT THE FORCES IN THE REINFORCEMENT
(Portion located inside or outside the failure surface)
TENSION SHEAR
IPTR = 0 IPSH = 0 No calculated forces
IPTR = 1 IPSH = 1 Calculation along the external portion imposed
IPTR = 2 IPSH = 2 Calculation along the outside = min(inside
outside)
IPTR = 3 IPSH = 3 Calculation along the inside = min(inside
outside)
IPSH = 4 Long inclusion
calculated along both sides
IPSH = 5 SHEAR is imposed

- UNITS : kN
m and degrees

XO	YO	RADIUS	OVER MOMENT	F-SOIL	F-SURCH	F-TOTAL	METH	INT
87.00	27.00	31.52	17390.44	.74	.74	5.57	2	*
87.00	27.00	521.52	17390.44	999.00	999.00	999.00	2	
94.00	27.00	37.42	14723.43	.86	.86	5.91	2	*
94.00	27.00	527.42	14723.43	999.00	999.00	999.00	2	
101.00	27.00	43.65	12051.20	1.06	1.06	6.18	2	*
101.00	27.00	533.65	12051.20	999.00	999.00	999.00	2	
108.00	27.00	50.09	9381.28	1.46	1.46	6.45	2	*
108.00	27.00	540.09	9381.28	999.00	999.00	999.00	2	
115.00	27.00	56.66	6711.14	2.19	2.19	6.70	2	*
115.00	27.00	546.66	6711.14	999.00	999.00	999.00	2	
122.00	27.00	63.32	4041.08	3.88	3.88	7.91	2	*
122.00	27.00	553.32	4041.08	999.00	999.00	999.00	2	
87.00	34.00	36.06	53664.71	.57	.57	2.65	2	*
87.00	34.00	526.06	53664.71	999.00	999.00	999.00	2	
94.00	34.00	41.32	35665.82	.64	.64	4.36	2	*
94.00	34.00	531.32	35665.82	999.00	999.00	999.00	2	
101.00	34.00	47.04	25664.56	.76	.76	6.63	2	*
101.00	34.00	537.04	25664.56	999.00	999.00	999.00	2	
108.00	34.00	53.06	22752.66	.84	.84	7.01	2	*
108.00	34.00	543.06	22752.66	999.00	999.00	999.00	2	
115.00	34.00	59.30	20085.37	.96	.96	7.19	2	*
115.00	34.00	549.30	20085.37	999.00	999.00	999.00	2	
122.00	34.00	65.70	17413.82	1.12	1.12	7.31	2	*
122.00	34.00	555.70	17413.82	999.00	999.00	999.00	2	
87.00	41.00	41.29	122871.10	.52	.52	1.87	2	*
87.00	41.00	531.29	122871.10	999.00	999.00	999.00	2	

94.00	41.00	45.96	95969.34	.50	.50	2.46	2	•
94.00	41.00	535.96	95969.34	999.00	999.00	999.00	2	
101.00	41.00	51.16	72521.05	.53	.53	3.40	2	•
101.00	41.00	541.16	72521.05	999.00	999.00	999.00	2	
108.00	41.00	56.75	51376.48	.60	.60	4.90	2	•
108.00	41.00	546.75	51376.48	999.00	999.00	999.00	2	
115.00	41.00	62.62	35792.70	.74	.74	9.12	2	•
115.00	41.00	552.62	35792.70	999.00	999.00	999.00	2	
122.00	41.00	68.71	30789.62	.83	.83	7.79	2	•
122.00	41.00	558.71	30789.62	999.00	999.00	999.00	2	
87.00	48.00	46.99	212603.50	.55	.55	1.46	2	•
87.00	48.00	536.99	212603.50	999.00	999.00	999.00	2	
94.00	48.00	51.14	175100.60	.50	.50	1.77	2	•
94.00	48.00	541.14	175100.60	999.00	999.00	999.00	2	
101.00	48.00	55.86	143606.20	.47	.47	2.24	2	•
101.00	48.00	545.86	143606.20	999.00	999.00	999.00	2	
108.00	48.00	61.02	116324.00	.47	.47	2.95	2	•
108.00	48.00	551.02	116324.00	999.00	999.00	999.00	2	
115.00	48.00	66.52	92046.00	.50	.50	4.00	2	•
115.00	48.00	556.52	92046.00	999.00	999.00	999.00	2	
122.00	48.00	72.27	69837.98	.56	.56	5.71	2	•
122.00	48.00	562.27	69837.98	999.00	999.00	999.00	2	
87.00	55.00	53.00	324980.00	.59	.59	1.18	2	•
87.00	55.00	543.00	324980.00	999.00	999.00	999.00	2	
94.00	55.00	56.71	274613.80	.53	.53	1.40	2	•
94.00	55.00	546.71	274613.80	999.00	999.00	999.00	2	
101.00	55.00	61.00	231770.80	.48	.48	1.67	2	•
101.00	55.00	551.00	231770.80	999.00	999.00	999.00	2	
108.00	55.00	65.76	195737.70	.45	.45	2.04	2	•
108.00	55.00	555.76	195737.70	999.00	999.00	999.00	2	
115.00	55.00	70.89	164425.90	.45	.45	2.60	2	•
115.00	55.00	560.89	164425.90	999.00	999.00	999.00	2	
122.00	55.00	76.32	136755.40	.45	.45	3.35	2	•
122.00	55.00	566.32	136755.40	999.00	999.00	999.00	2	

Fichier de donnees : c:\SOIL NAILED WALL DESIGN 3.rel
Date :Apr 9 2003
Time :13h 14mn 55s

Name of data file :c:\SOIL NAILED WALL DESIGN 3.tal

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|-----------------------------------------------------|
|                                                     |
|                 TALREN 97 program                   |
|                                                     |
|       (TALus RENForces = reinforced slopes)         |
|                                                     |
|                        -o-                          |
|                                                     |
|             TALREN 97 1.1  of  02/15/98             |
|                                                     |
|     Computer program for the stability analysis     |
|                of reinforced slopes                 |
|                                                     |
|                        -o-                          |
|                                                     |
|       Copyright (c) 1981 TALREN - TERRASOL          |
|-----------------------------------------------------|
| |-----------|        Developed  by TERRASOL         |
| |           |                                       |
| |-|   /----|         IMMEUBLE HELIOS                 |
|   |  / /---|         72 Avenue PASTEUR               |
|   |--/ /    |    93108 MONTREUIL cedex - FRANCE     |
| |-----/   /-|                                       |
| |        /      Telephone : 00 33 1 49 88 24 42     |
| |-------/                                           |
|-----------------------------------------------------|
```

Project number ..:WALL 1
Project location :
Title ....:STABILITY OF SOIL NAILED WALL

Comment(s) :
- KM 15+060 - KM 15+187
- H = 27 m

DATA - DATA - DATA - DATA - DATA - DATA - DATA - DATA - DATA

* ANALYSIS METHOD:BISHOP

- Initial value of F: 1.00
- No zoning for hydraulic data
- Number of subdivisions for the failure surface: 49

* FAILURE SURFACE: CIRCULAR

- XO = 102.00 YO = 51.00 DX = 5.00 DY = 5.00 AX = .0 AY = .0 NX = 6 NY = 6
- Maximum number for circles for the calculation 10
- Radius increment : 5.0
- Point of passage for the first circle: X = 61.420 Y = 8.580
- Minimum X value for the second intersection point of the failure surface with the slope: .00

* GEOMETRICAL DATA DEFINED BY POINTS AND SEGMENTS

POINT	X	Y
1	.00	70.00
2	27.84	47.87
3	36.42	47.87
4	51.42	36.10
5	56.00	20.71
6	59.00	20.71
7	61.42	8.58
8	70.00	8.58
9	.00	50.00
10	10.71	40.71
11	17.13	33.58
12	24.29	28.58
13	34.26	24.29
14	43.58	22.13

SEG No	ENDPT 1	ENDPT 2	UNDER SOIL
1	1	2	1
2	2	3	1
3	3	4	1
4	4	5	1
5	5	6	2
6	6	7	2
7	7	8	2
8	9	10	2
9	10	11	2
10	11	12	2
11	12	13	2
12	13	14	2
13	14	5	2

SLOPE SEGMENT NO
1
2
3
4
5
6
7

* SOIL CHARACTERISTICS

SOIL	GAMMA	FS1	COHES c	Dc/z	Fc	PHI	Fphi	RU	QS	PL	KSB
1	18.0	1.05	.0	.00	1.00	30.0	1.20	.00	70.0	800.0	28000.0
2	22.0	1.05	30.0	.00	1.50	38.0	1.20	.00	200.0	1700.0	65000.0

* REINFORCEMENT DATA

No.	TYPE	STRENGTH	SPAC.	X	Y	L	ANG	LB	DA	RAL	REQ	IND	SHEAR	L MIN	MMAX	EI	CA	IENC
1	NAIL	301.4	1.00	51.61	35.35	24.0	15.0	2.00	20.0	.0	.063	1	.0	.00	.0	.0	.0	0
2	NAIL	301.4	1.00	51.98	33.85	24.0	15.0	2.00	20.0	.0	.063	1	.0	.00	.0	.0	.0	0
3	NAIL	301.4	1.00	52.40	32.35	24.0	15.0	2.00	20.0	.0	.063	1	.0	.00	.0	.0	.0	0
4	NAIL	301.4	1.00	52.85	30.85	24.0	15.0	2.00	20.0	.0	.063	1	.0	.00	.0	.0	.0	0
5	NAIL	301.4	1.00	53.18	29.35	24.0	15.0	2.00	20.0	.0	.063	1	.0	.00	.0	.0	.0	0
6	NAIL	301.4	1.00	53.60	27.85	16.0	15.0	2.00	20.0	.0	.063	1	.0	.00	.0	.0	.0	0
7	NAIL	301.4	1.00	54.20	26.35	16.0	15.0	2.00	20.0	.0	.063	1	.0	.00	.0	.0	.0	0
8	NAIL	301.4	1.00	54.60	24.85	16.0	15.0	2.00	20.0	.0	.063	1	.0	.00	.0	.0	.0	0
9	NAIL	301.4	1.00	55.00	23.35	12.0	15.0	2.00	20.0	.0	.063	1	.0	.00	.0	.0	.0	0
10	NAIL	301.4	1.00	55.50	21.85	12.0	15.0	2.00	20.0	.0	.063	1	.0	.00	.0	.0	.0	0
11	NAIL	698.0	2.00	59.10	19.71	16.0	15.0	2.00	20.0	.0	.063	1	.0	.00	.0	.0	.0	0
12	NAIL	698.0	2.00	59.50	17.71	16.0	15.0	2.00	20.0	.0	.063	1	.0	.00	.0	.0	.0	0
13	NAIL	698.0	2.00	60.00	15.71	16.0	15.0	2.00	20.0	.0	.063	1	.0	.00	.0	.0	.0	0
14	NAIL	698.0	2.00	60.40	13.71	12.0	15.0	2.00	20.0	.0	.063	1	.0	.00	.0	.0	.0	0
15	NAIL	698.0	2.00	60.75	11.71	12.0	15.0	2.00	20.0	.0	.063	1	.0	.00	.0	.0	.0	0
16	NAIL	698.0	2.00	61.30	9.71	8.0	15.0	2.00	20.0	.0	.063	1	.0	.00	.0	.0	.0	0

- CALCULATION TYPE: - For NAILS: IND = 1 Tension calculated / Shear imposed
IND = 2 Zero tension / Shear calculated
IND = 3 Tension and Shear calculated

- For ANCHORS: IND = 1 Pull-out resistance as a function of the length beyond the failure surface
IND = 2 Pull-out resistance either total or zero depending on the position of the fictitious a

- Taking into account the portion of the reinforcement inside or outside the failure surface

SHEAR		TENSION
. IENC = 0 :	External/Internal	External
. IENC = 1 :	External	External
. IENC = 2 :	External/Internal	External/Internal
. IENC = 3 :	External	External/Internal

- The increase in normal stresses along the failure surface induced by the reinforcement is calculated by the program

* PARTIAL SAFETY FACTORS FOR THE STRENGTH PARAMETERS

- Soil/inclusion lateral friction
. Nail....................FqsNa : 1.400
. Anchor ....FqsAn : 1.000
. Reinforcing strip......FqsRS : 1.000
- Limit pressure of soils..................Fpl : 2.000
- Intrinsic strength of the inclusions
. Nail ....FaNai : 1.150
. Anchor ....FaAnc : 1.000
. Reinforcing strip......FaRS : 1.000
. Brace ....FaBra : 1.000
- Calculation method ....Fs3 : 1.125

Fichier de données : c:\SOIL NAILED WALL DESIGN 3.re2
Date : Apr 9 2003
Time : 13h 14mn 55s

Name of data file : c:\SOIL NAILED WALL DESIGN 3.tal

* RESULTS - RESULTS - RESULTS - RESULTS - RESULTS - RESULTS - RESULTS - RESULTS - RESULTS *

Indicators (*) - INT : There are other intersections between the failure surface and the slope
- For the safety factors
it indicates that convergence was not obtained

METHOD (analysis method): 1 - FELLENIUS
2 - BISHOP
3 - PERTURBATIONS
- The overturning moments are computed taking into account the defined loads

- LIMIT FORCES IN THE REINFORCEMENT :
. TENSION
by the STEEL STRENGTH if ITR = 1
by qs if ITR = 2
. SHEAR
Long inclusion/high bending stiffness if ISHR = 1
Long inclusion/low bending stiffness if ISHR = 2
Short inclusion/high bending stiffness if ISHR = 3
Short inclusion/low bending stiffness if ISHR = 4
Limited by TR/2 if ISHR = 5

- INDICATOR FOR TAKING INTO ACCOUNT THE FORCES IN THE REINFORCEMENT
(Portion located inside or outside the failure surface)
TENSION SHEAR
IPTR = 0 IPSH = 0 No calculated forces
IPTR = 1 IPSH = 1 Calculation along the external portion imposed
IPTR = 2 IPSH = 2 Calculation along the outside = min(inside outside)
IPTR = 3 IPSH = 3 Calculation along the inside = min(inside outside)
IPSH = 4 Long inclusion
calculated along both sides
IPSH = 5 SHEAR is imposed

- UNITS : kN
m and degrees

XO	YO	RADIUS	OVER MOMENT	F-SOIL	F-SURCH	F-TOTAL	METH	INT
102.00	51.00	58.70	174538.90	.47	.47	2.02	2	*
102.00	51.00	548.70	174536.90	999.00	999.00	999.00	2	
107.00	51.00	62.26	152470.90	.46	.46	2.40	2	*
107.00	51.00	552.26	152470.90	999.00	999.00	999.00	2	
112.00	51.00	66.01	132689.90	.46	.46	2.87	2	*
112.00	51.00	556.01	132689.90	999.00	999.00	999.00	2	
117.00	51.00	69.92	114168.70	.46	.46	3.52	2	*
117.00	51.00	559.92	114168.70	999.00	999.00	999.00	2	
122.00	51.00	73.95	97102.56	.50	.50	4.44	2	*
122.00	51.00	563.95	97102.56	999.00	999.00	999.00	2	
127.00	51.00	78.10	81066.84	.54	.54	5.69	2	*
127.00	51.00	568.10	81066.84	999.00	999.00	999.00	2	
102.00	56.00	62.41	240149.70	.48	.48	1.66	2	*
102.00	56.00	552.41	240149.70	999.00	999.00	999.00	2	
107.00	56.00	65.77	213360.50	.46	.46	1.90	2	*
107.00	56.00	555.77	213360.50	999.00	999.00	999.00	2	
112.00	56.00	69.33	189269.90	.45	.45	2.22	2	*
112.00	56.00	559.33	189269.90	999.00	999.00	999.00	2	
117.00	56.00	73.06	167371.50	.45	.45	2.64	2	*
117.00	56.00	563.06	167371.50	999.00	999.00	999.00	2	
122.00	56.00	76.93	147274.20	.45	.45	3.16	2	*
122.00	56.00	566.93	147274.20	999.00	999.00	999.00	2	
127.00	56.00	80.93	128717.20	.46	.46	3.84	2	*
127.00	56.00	570.93	128717.20	999.00	999.00	999.00	2	
102.00	61.00	66.29	316226.80	.50	.50	1.41	2	*
102.00	61.00	556.29	316226.80	999.00	999.00	999.00	2	

107.00	61.00	69.46	283335.00	.47	.47	1.59	2	*
107.00	61.00	559.46	283335.00	999.00	999.00	999.00	2	
112.00	61.00	72.84	253966.40	.45	.45	1.80	2	*
112.00	61.00	562.84	253966.40	999.00	999.00	999.00	2	
117.00	61.00	76.40	227967.90	.45	.45	2.09	2	*
117.00	61.00	566.40	227967.90	999.00	999.00	999.00	2	
122.00	61.00	80.11	203900.90	.44	.44	2.43	2	*
122.00	61.00	570.11	203900.90	999.00	999.00	999.00	2	
127.00	61.00	83.95	182360.80	.43	.43	2.84	2	*
127.00	61.00	573.95	182360.80	999.00	999.00	999.00	2	
102.00	66.00	70.31	400019.50	.52	.52	1.25	2	*
102.00	66.00	560.31	400019.50	999.00	999.00	999.00	2	
107.00	66.00	73.31	363353.50	.50	.50	1.39	2	*
107.00	66.00	563.31	363353.50	999.00	999.00	999.00	2	
112.00	66.00	76.52	328297.60	.47	.47	1.54	2	*
112.00	66.00	566.52	328297.60	999.00	999.00	999.00	2	
117.00	66.00	79.91	296489.20	.45	.45	1.73	2	*
117.00	66.00	569.91	296489.20	999.00	999.00	999.00	2	
122.00	66.00	83.47	268256.30	.44	.44	1.96	2	*
122.00	66.00	573.47	268256.30	999.00	999.00	999.00	2	
127.00	66.00	87.16	242473.30	.43	.43	2.24	2	*
127.00	66.00	577.16	242473.30	999.00	999.00	999.00	2	
102.00	71.00	74.45	483957.30	.55	.55	1.14	2	*
102.00	71.00	564.45	483957.30	999.00	999.00	999.00	2	
107.00	71.00	77.29	447442.30	.51	.51	1.24	2	*
107.00	71.00	567.29	447442.30	999.00	999.00	999.00	2	
112.00	71.00	80.34	410759.60	.49	.49	1.36	2	*
112.00	71.00	570.34	410759.60	999.00	999.00	999.00	2	
117.00	71.00	83.57	374700.90	.47	.47	1.48	2	*
117.00	71.00	573.57	374700.90	999.00	999.00	999.00	2	
122.00	71.00	86.98	340385.10	.45	.45	1.65	2	*
122.00	71.00	576.98	340385.10	999.00	999.00	999.00	2	
127.00	71.00	90.53	309875.50	.44	.44	1.87	2	*
127.00	71.00	580.53	309875.50	999.00	999.00	999.00	2	
102.00	76.00	78.69	568609.40	.58	.58	1.06	2	*
102.00	76.00	568.69	568609.40	999.00	999.00	999.00	2	
107.00	76.00	81.38	531349.60	.54	.54	1.13	2	*
107.00	76.00	571.38	531349.60	999.00	999.00	999.00	2	
112.00	76.00	84.28	494868.30	.51	.51	1.23	2	*
112.00	76.00	574.28	494868.30	999.00	999.00	999.00	2	
117.00	76.00	87.37	458158.70	.48	.48	1.32	2	*
117.00	76.00	577.37	458158.70	999.00	999.00	999.00	2	
122.00	76.00	90.64	421955.70	.46	.46	1.44	2	*
122.00	76.00	580.64	421955.70	999.00	999.00	999.00	2	
127.00	76.00	94.05	385820.50	.45	.45	1.60	2	*
127.00	76.00	584.05	385820.50	999.00	999.00	999.00	2	

Fichier de donnees : c:\SEISMIC 1.rel
Date :Apr 9 2003
Time :14h 57mn 35s

Name of data file :c:\SEISMIC 1.tal

TALREN 97 program

(TALus RENForces = reinforced slopes)

-o-

TALREN 97 1.1 of 02/15/98

Computer program for the stability analysis
of reinforced slopes

-o-

Developed by TERRASOL

IMMEUBLE HELIOS
72 Avenue PASTEUR
93108 MONTREUIL cedex - FRANCE

Telephone : 00 33 1 49 88 24 42

Project number ...:WALL 1
Project location :
Title ....:STABILITY OF SOIL NAILED WALL

Comment(s) :
- KM 15+060 - KM 15+187
- H = 27 m

DATA - DATA - DATA - DATA - DATA - DATA - DATA - DATA - DATA

* ANALYSIS METHOD:BISHOP

- Initial value of F: 1.00
- No zoning for hydraulic data
- Number of subdivisions for the failure surface: 49

* FAILURE SURFACE: CIRCULAR

- XO = 38.00 YO = 37.00 DX = 5.00 DY = 5.00 AX = 5.0 AY = .0 NX = 4 NY = 3
- Maximum number for circles for the calculation 10
- Radius increment : 5.0
- Point of passage for the first circle: X = 61.400 Y = 8.440
- Minimum X value for the second intersection point of the failure surface with the slope: .00

* GEOMETRICAL DATA DEFINED BY POINTS AND SEGMENTS

POINT	X	Y
1	.00	70.00
2	27.84	47.87
3	36.42	47.87
4	51.42	36.10
5	56.00	20.71
6	59.00	20.71
7	61.42	8.58
8	90.00	8.58
9	.00	50.00
10	10.71	40.71
11	17.13	33.58
12	24.29	28.56
13	34.26	24.29
14	43.58	22.13

SEG No	ENDPT 1	ENDPT 2	UNDER SOIL
1	1	2	1
2	2	3	1
3	3	4	1
4	4	5	1
5	5	6	2
6	6	7	2
7	7	8	2
8	9	10	2
9	10	11	2
10	11	12	2
11	12	13	2
12	13	14	2
13	14	5	2

SLOPE SEGMENT NO
1
2
3
4
5
6
7

* SOIL CHARACTERISTICS

SOIL	GAMMA	FS1	COHES c	Dc/z	Fc	PHI	Fphi	RU	QS	PL	KSB
1	18.0	1.05	.0	.00	1.00	30.0	1.20	.00	70.0	800.0	28000.0
2	22.0	1.05	30.0	.00	1.50	38.0	1.20	.00	200.0	1700.0	65000.0

* SEISMIC SIMULATION

- Horizontal acceleration: .27 x g (positive to the right)
- vertical acceleration : .00 x g (positive downwards)

The horizontal seismic acceleration is not applied
to surcharges and water located outside the slope !!!

* REINFORCEMENT DATA

No.	TYPE	STRENGTH	SPAC.	X	Y	L	ANG	LB	DA	RAL	REQ	IND	SHEAR	L MIN	MMAX	EI	CA	IENC
1	NAIL	301.4	1.00	51.61	35.35	24.0	15.0	2.00	20.0	.0	.063	1	.0	.00	.0	.0	.0	0
2	NAIL	301.4	1.00	51.98	33.85	24.0	15.0	2.00	20.0	.0	.063	1	.0	.00	.0	.0	.0	0
3	NAIL	301.4	1.00	52.40	32.35	24.0	15.0	2.00	20.0	.0	.063	1	.0	.00	.0	.0	.0	0
4	NAIL	301.4	1.00	52.85	30.85	24.0	15.0	2.00	20.0	.0	.063	1	.0	.00	.0	.0	.0	0
5	NAIL	301.4	1.00	53.18	29.35	24.0	15.0	2.00	20.0	.0	.063	1	.0	.00	.0	.0	.0	0
6	NAIL	301.4	1.00	53.60	27.85	16.0	15.0	2.00	20.0	.0	.063	1	.0	.00	.0	.0	.0	0
7	NAIL	301.4	1.00	54.20	26.35	16.0	15.0	2.00	20.0	.0	.063	1	.0	.00	.0	.0	.0	0
8	NAIL	301.4	1.00	54.60	24.85	16.0	15.0	2.00	20.0	.0	.063	1	.0	.00	.0	.0	.0	0
9	NAIL	301.4	1.00	55.00	23.35	12.0	15.0	2.00	20.0	.0	.063	1	.0	.00	.0	.0	.0	0
10	NAIL	301.4	1.00	55.50	21.85	12.0	15.0	2.00	20.0	.0	.063	1	.0	.00	.0	.0	.0	0
11	NAIL	698.0	2.00	59.10	19.71	16.0	15.0	2.00	20.0	.0	.063	1	.0	.00	.0	.0	.0	0
12	NAIL	696.0	2.00	59.50	17.71	16.0	15.0	2.00	20.0	.0	.063	1	.0	.00	.0	.0	.0	0
13	NAIL	698.0	2.00	60.00	15.71	16.0	15.0	2.00	20.0	.0	.063	1	.0	.00	.0	.0	.0	0
14	NAIL	698.0	2.00	60.40	13.71	12.0	15.0	2.00	20.0	.0	.063	1	.0	.00	.0	.0	.0	0
15	NAIL	698.0	2.00	60.75	11.71	12.0	15.0	2.00	20.0	.0	.063	1	.0	.00	.0	.0	.0	0
16	NAIL	698.0	2.00	61.30	9.71	8.0	15.0	2.00	20.0	.0	.063	1	.0	.00	.0	.0	.0	0

- CALCULATION TYPE: - For NAILS: IND = 1 Tension calculated / Shear imposed
IND = 2 Zero tension / Shear calculated
IND = 3 Tension and Shear calculated

- For ANCHORS: IND = 1 Pull-out resistance as a function of the length beyond the failure surface
IND = 2 Pull-out resistance either total or zero depending on the position of the fictitious a

- Taking into account the portion of the reinforcement inside or outside the failure surface

SHEAR		TENSION
. IENC = 0 :	External/Internal	External
. IENC = 1 :	External	External
. IENC = 2 :	External/Internal	External/Internal
. IENC = 3 :	External	External/internal

- The increase in normal stresses along the failure surface induced by the reinforcement
is calculated by the program

* PARTIAL SAFETY FACTORS FOR THE STRENGTH PARAMETERS

- Soil/inclusion lateral friction
. Nail...................FqsNa : 1.400
. Anchor ....FqsAn : 1.000
. Reinforcing strip......FqsRS : 1.000
- Limit pressure of soils..................Fpl : 2.000
- Intrinsic strength of the inclusions
. Nail ....FaNai : 1.150
. Anchor ....FaAnc : 1.000
. Reinforcing strip......FaRS : 1.000
. Brace ....FaBra : 1.000
- Calculation method ....Fs3 : 1.125

38.00	47.00	535.10	1221112.00	999.00	999.00	999.00	2	
42.96	47.44	533.12	1221112.00	999.00	999.00	999.00	2	
47.96	47.87	41.65	866953.90	1.18	1.18	1.18	2	
47.96	47.87	46.65	1274799.00	1.21	1.21	1.21	2	
47.96	47.87	531.65	1274799.00	999.00	999.00	999.00	2	
52.94	48.31	40.75	721025.90	1.03	1.03	1.03	2	
52.94	48.31	45.75	1089026.00	1.10	1.10	1.10	2	
52.94	48.31	50.75	1550237.00	1.19	1.19	1.19	2	
52.94	48.31	530.75	1550237.00	999.00	999.00	999.00	2	

SLOPE SEGMENT NO
1
2
3
4
5
6
7

* SOIL CHARACTERISTICS
****

SOIL	GAMMA	FS1	COHES c	Dc/z	Fc	PHI	Fphi	RU	QS	PL	KSB
1	18.0	1.05	.0	.00	1.00	30.0	1.20	.00	70.0	800.0	28000.0
2	22.0	1.05	30.0	.00	1.50	38.0	1.20	.00	200.0	1700.0	65000.0

* SEISMIC SIMULATION
****

- Horizontal acceleration: .49 x g (positive to the right)
- vertical acceleration : .00 x g (positive downwards)

The horizontal seismic acceleration is not applied
to surcharges and water located outside the slope !!!

* REINFORCEMENT DATA
****

No.	TYPE	STRENGTH	SPAC.	X	Y	L	ANG	LB	DA	RAL	REQ	IND	SHEAR	L MIN	MMAX	EI	CA	IENC
1	NAIL	301.4	1.00	51.61	35.35	24.0	15.0	2.00	20.0	.0	.063	1	.0	.00	.0	.0	.0	0
2	NAIL	301.4	1.00	51.98	33.85	24.0	15.0	2.00	20.0	.0	.063	1	.0	.00	.0	.0	.0	0
3	NAIL	301.4	1.00	52.40	32.35	24.0	15.0	2.00	20.0	.0	.063	1	.0	.00	.0	.0	.0	0
4	NAIL	301.4	1.00	52.85	30.85	24.0	15.0	2.00	20.0	.0	.063	1	.0	.00	.0	.0	.0	0
5	NAIL	301.4	1.00	53.18	29.35	24.0	15.0	2.00	20.0	.0	.063	1	.0	.00	.0	.0	.0	0
6	NAIL	301.4	1.00	53.60	27.85	16.0	15.0	2.00	20.0	.0	.063	1	.0	.00	.0	.0	.0	0
7	NAIL	301.4	1.00	54.20	26.35	16.0	15.0	2.00	20.0	.0	.063	1	.0	.00	.0	.0	.0	0
8	NAIL	301.4	1.00	54.60	24.85	16.0	15.0	2.00	20.0	.0	.063	1	.0	.00	.0	.0	.0	0
9	NAIL	301.4	1.00	55.00	23.35	12.0	15.0	2.00	20.0	.0	.063	1	.0	.00	.0	.0	.0	0
10	NAIL	301.4	1.00	55.50	21.85	12.0	15.0	2.00	20.0	.0	.063	1	.0	.00	.0	.0	.0	0
11	NAIL	698.0	2.00	59.10	19.71	16.0	15.0	2.00	20.0	.0	.063	1	.0	.00	.0	.0	.0	0
12	NAIL	698.0	2.00	59.50	17.71	16.0	15.0	2.00	20.0	.0	.063	1	.0	.00	.0	.0	.0	0
13	NAIL	698.0	2.00	60.00	15.71	16.0	15.0	2.00	20.0	.0	.063	1	.0	.00	.0	.0	.0	0
14	NAIL	698.0	2.00	60.40	13.71	12.0	15.0	2.00	20.0	.0	.063	1	.0	.00	.0	.0	.0	0
15	NAIL	698.0	2.00	60.75	11.71	12.0	15.0	2.00	20.0	.0	.063	1	.0	.00	.0	.0	.0	0
16	NAIL	698.0	2.00	61.30	9.71	8.0	15.0	2.00	20.0	.0	.063	1	.0	.00	.0	.0	.0	0

- CALCULATION TYPE: - For NAILS; IND = 1 Tension calculated / Shear imposed
IND = 2 Zero tension / Shear calculated
IND = 3 Tension and Shear calculated

- For ANCHORS: IND = 1 Pull-out resistance as a function of the length beyond the failure surface
IND = 2 Pull-out resistance either total or zero depending on the position of the fictitious a

- Taking into account the portion of the reinforcement inside or outside the failure surface

SHEAR	TENSION
. IENC = 0 : External/Internal	External
. IENC = 1 : External	External
. IENC = 2 : External/Internal	External/Internal
. IENC = 3 : External	External/internal

- The increase in normal stresses along the failure surface induced by the reinforcement
is calculated by the program

* PARTIAL SAFETY FACTORS FOR THE STRENGTH PARAMETERS
****

- Soil/inclusion lateral friction
. Nail....................FqsNa : 1.400
. Anchor ....FqsAn : 1.000
. Reinforcing strip......FqsRS : 1.000
- Limit pressure of soils..................Fpl : 2.000
- Intrinsic strength of the inclusions
. Nail ....FaNai : 1.150
. Anchor ....FaAnc : 1.000
. Reinforcing strip......FaRS : 1.000
. Brace ....FaBra : 1.000
- Calculation method ....Fs3 : 1.125

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Fichier de donnees : c:\SEISMIC 2.ta2                    Name of data file  c:\SEISMIC 2.tal
Date  :Apr  9 2003
Time  :15h 43mn 45s

*********************************************************************************************
*
*  RESULTS  -  RESULTS  -  RESULTS  -  RESULTS  -  RESULTS  -  RESULTS  -  RESULTS  -  RESULTS  -  RESULTS  *
*
*********************************************************************************************

Indicators (*) - INT : There are other intersections between the failure surface and the slope
- For the safety factors
it indicates that convergence was not obtained

METHOD (analysis method): 1 - FELLENIUS
2 - BISHOP
3 - PERTURBATIONS
- The overturning moments are computed taking into account the defined loads

- LIMIT FORCES IN THE REINFORCEMENT :
. TENSION
by the STEEL STRENGTH            if ITR = 1
by qs                            if ITR = 2
. SHEAR
Long inclusion/high bending stiffness    if ISHR = 1
Long inclusion/low bending stiffness     if ISHR = 2
Short inclusion/high bending stiffness   if ISHR = 3
Short inclusion/low bending stiffness    if ISHR = 4
limited by TR/2                          if ISHR = 5

- INDICATOR FOR TAKING INTO ACCOUNT THE FORCES IN THE REINFORCEMENT
(Portion located inside or outside the failure surface)
TENSION    SHEAR
IPTR = 0   IPSH = 0  No calculated forces
IPTR = 1   IPSH = 1  Calculation along the external portion imposed
IPTR = 2   IPSH = 2  Calculation along the outside = min(inside
outside)
IPTR = 3   IPSH = 3  Calculation along the inside  = min(inside
outside)
IPSH = 4  Long inclusion
calculated along both sides
IPSH = 5  SHEAR is imposed

- UNITS : kN
m and degrees
```


XO	YO	RADIUS	OVER MOMENT	F-SOIL	F-SURCH	F-TOTAL	METH	INT
60.00	16.00	7.68	9785.67	1.46	1.46	2.04	2	
60.00	16.00	12.67	43386.61	1.69	1.69	1.70	2	
60.00	16.00	497.68	43386.61	999.00	999.00	999.00	2	
65.00	16.00	8.37	6486.54	1.07	1.07	2.53	2	
65.00	16.00	496.37	6486.54	999.00	999.00	999.00	2	
70.00	16.00	501.45	6486.54	999.00	999.00	999.00	2	
75.00	16.00	505.56	6486.54	999.00	999.00	999.00	2	
60.00	21.00	17.63	39349.20	1.07	1.07	1.46	2	
60.00	21.00	17.63	109016.80	1.25	1.25	1.25	2	
60.00	21.00	502.63	109016.80	999.00	999.00	999.00	2	
65.00	21.00	13.06	26030.96	.80	.80	1.60	2	
65.00	21.00	503.06	26030.96	999.00	999.00	999.00	2	
70.00	21.00	505.22	26030.96	999.00	999.00	999.00	2	
75.00	21.00	508.32	26030.96	999.00	999.00	999.00	2	
60.00	26.00	17.61	93669.30	.95	.95	1.13	2	
60.00	26.00	22.61	207732.90	1.05	1.05	1.06	2	
60.00	26.00	507.61	207732.90	999.00	999.00	999.00	2	
65.00	26.00	17.92	69320.04	.70	.70	1.23	2	
65.00	26.00	507.92	69320.04	999.00	999.00	999.00	2	
70.00	26.00	509.55	69320.04	999.00	999.00	999.00	2	
75.00	26.00	512.21	69320.04	999.00	999.00	999.00	2	

BIOGRAPHY

Nurhan Ecemis was born in Istanbul in 1979. She completed her high school education in Kultur Fen Lisesi in 1997. In 1997, she began her undergraduate studies in the Civil Engineering Department at the Trakya University, Edirne. In 1999, she transferred to Istanbul Kultur University with a scholarship. She enrolled Istanbul Kultur University, faculty of Engineering in 2001 for a civil engineering degree (B.Sc.). She graduated in the June term of the academic year 2000-2001. Furthermore, she has been awarded with a “High Honor” certificate from the university.

In 2001, she commenced her post-graduate studies in Istanbul Technical University, as a research student in Geotechnical Engineering.